JOURNAL OF
THE INSTITUTION OF
CIVIL ENGINEERS.

No. 2. 1940-41.
DECEMBER 1940.

Paper No. 5226.

"The Graphical Solution of Portal Frames."

By JEFFREY WILLIAM HITCHEN KING, M.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION.

THE analytical solution of stiff-jointed portal frames is, in general, a complex mathematical problem, and even the usual methods of graphical solution by graphical integration are apt to be laborious, especially in cases where the feet of the portal are fixed, or where the frame is a complete rectangle. This Paper is intended to present an extension of the use of "characteristic points" to the solution of unsymmetrical portal frames. The method of solution known as "characteristic points" was originated by Professor T. Claxton Fidler,[2] M. Inst. C.E., in 1883. The solutions of continuous-beam problems, using this method, are dealt with in many modern textbooks, of which the most notable, in the Author's opinion, is that of Dr. E. H. Salmon, M. Inst. C.E.[3] Since, however, few works, apart from those of Dr. Salmon [4, 5], discuss the case where the supports are allowed to settle, a short proof is given here.

---

[1] Correspondence on this Paper can be accepted until the 15th April, 1941, and will be published in the Institution Journal for October 1941.—SEC. INST. C.E.

[2] "Continuous Girder Bridges." Minutes of Proceedings Inst. C.E., vol. lxxiv (1882–83, Part iv), p. 209.

[3] E. H. Salmon, "Materials and Structures." Longmans, Green and Co., London, 1931.

[4] *Loc. cit.*, vol. 1, pp. 140 *et seq.*

[5] E. H. Salmon, "Characteristic Points." Selected Engineering Paper No. 46, Inst. C.E., 1927.

The following conventions will be used throughout the Paper :—

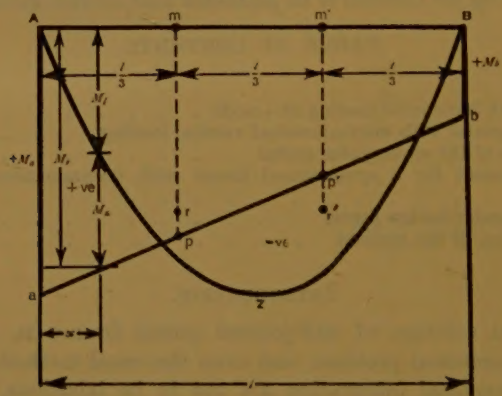
(*a*) Bending moment will be considered positive (+) when it causes the beam to arch upwards, or the portal to arch outwards ; curves indicating positive bending moment will be drawn above the base-line.

(*b*) Shear force will be considered positive (+) when the forces to the right of the section in question tend to lift that side.

(*c*) Loads and deflexions will be considered positive (+) when they act downwards or towards the inside of a portal frame.

The curve AZB in *Fig. 1* is the bending-moment diagram for a freely-supported beam subjected to a vertical loading. If restraining moments are introduced at A and B, of M_a and M_b respectively, and the span of the beam is l, then the restraining-moment diagram (hereafter referred to as

Fig. 1.



the R.M. diagram) becomes the base-line for the compound bending-moment diagram, which is made up of the negative bending-moment diagram AZB and the positive bending-moment diagram AabB, and represents the case of a restrained beam.

The bending moment at X (where AX = x) is :

$$M_x = M_r + M_l = EI \cdot \frac{d_2 y}{dx^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

If it is assumed that E and I are constant throughout the span, then, multiplying both sides by x, and integrating between the limits A and B, equation (1) becomes :

$$\frac{l^2}{6}(M_a + 2M_b) + C\bar{x} = EI(l\theta_b + y_a - y_b) \quad . \quad . \quad . \quad (2)$$

where $C\bar{x}$ denotes the moment of the bending-moment diagram about A, θ_b denotes the slope at B (positive downwards in the direction of x increas-

ing), and y_a, y_b denote the settlements of the ends A and B respectively. Replacing $y_b - y_a$ by Δ, equation (1) may be transformed to :

$$\frac{2EI}{l} \cdot \theta_b = \frac{2EI}{l^2}\Delta + \frac{(M_a+2M_b)}{3} + \frac{2C\bar{x}}{l^2}$$

$$= \frac{2EI\Delta + 2C\bar{x}}{l^2} + \mathrm{p'm'} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If m′r′ is made equal to $\frac{2EI\Delta + 2C\bar{x}}{l^2}$,

then $$\theta_b = \frac{l}{2EI}(\mathrm{p'r'}) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

It should be noted here that p′r′ is positive measured upwards, and l is positive when B is at the right-hand end of the span.

If B is now considered as an intermediate support instead of as an end support, a similar expression for θ_b may be obtained from the conditions of the adjoining span. By equating these two values for θ_b graphically, it is possible to draw the complete bending-moment diagram for a two-span continuous beam. Furthermore, since a similar relationship exists between all adjacent spans, it follows that it is possible to draw a similar diagram for any number of spans. Until recently, trial-and-error methods were largely used to obtain a diagram which would satisfy the conditions at all supports. This Paper will make use of a construction [1] which renders such tedious work unnecessary.

Dr. Salmon has shown [2] that, by drawing a distorted diagram in which spans were made proportional to $\frac{\text{length}}{EI}$ instead of to length, beams in which E and I were not constant throughout the length could be solved by the same methods as beams with a constant E and I. He has further shown that symmetrical portal frames, symmetrically loaded, or frames with their nodes restrained in position, could also be solved by the same methods, by opening them out and treating them as continuous beams ; loss of symmetry, however, or movement of the nodes, or both, would render such a method inoperative.

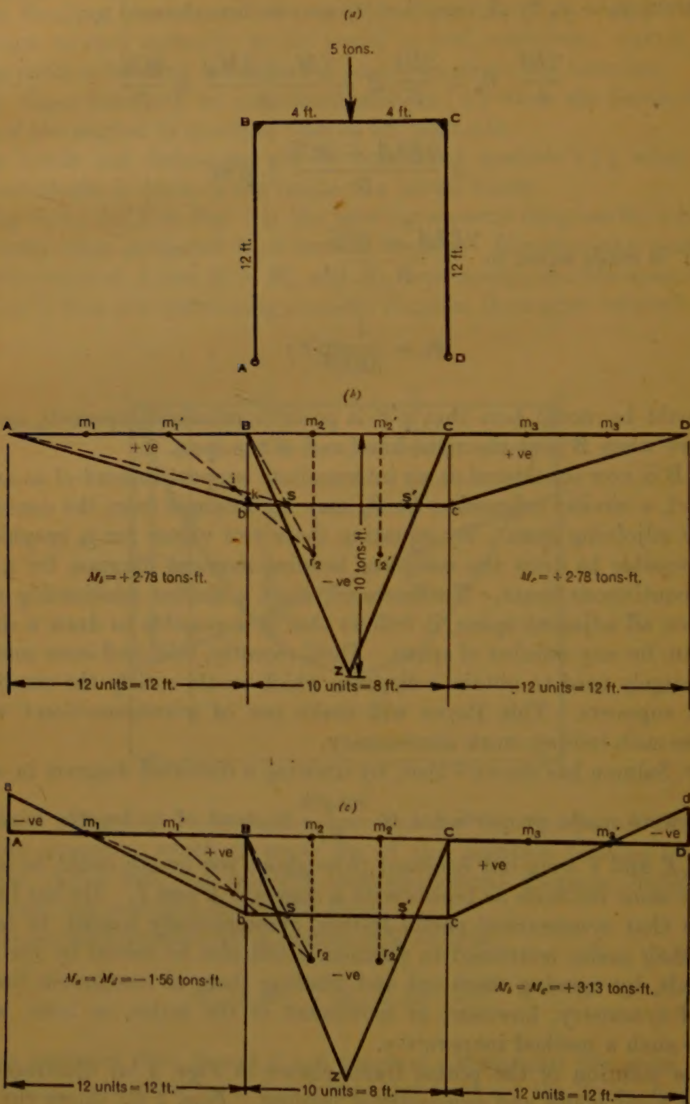
The solution of the portal frame shown in *Figs. 2* (*a*) illustrates the case of fixed nodes and symmetrical loading. *Figs. 2* (*b*) shows the construction for such a solution if the feet A and D (*Figs. 2* (*a*)) are hinged. It is supposed that the value of I is 100 for each side leg and 80 for the

---

[1] J. W. H. King, "Bending Moments in Continuous Beams." *Concrete and Constructional Engineering*, vol. 34, p. 265 (May 1939).

[2] See footnote (3), p. 85.

Figs. 2.



horizontal member. In *Figs. 2* (*b*) a straight line is drawn horizontally so that:

$$AB : BC : CD :: \frac{12}{100} : \frac{8}{80} : \frac{12}{100}$$

AB, BC, and CD are trisected in the pairs of points m_1m_1', m_2m_2', and

m_3m_3' respectively. On BC is erected the negative bending-moment diagram which would result from the same loading, 5 tons, applied at the centre of a freely-supported beam. m_2r_2 and $m_2'r_2'$ are drawn so that:

$$m_2r_2 = \frac{2}{l^2} \text{ (moment of the area of the bending-moment diagram BZC about C),}$$

and

$$m_2'r_2' = \frac{2}{l^2} \text{ (moment of the area of the bending-moment diagram BZC about B).}$$

Since in this case the load is centrally applied,

$$m_2r_2 = m_2'r_2' = -5 \text{ ton-feet.}$$

Furthermore, since there is no relative vertical movement of B and C, $\Delta = 0$, and so r_2 and r_2' are the characteristic points for the span BC. The points m_1, m_1', m_3, and m_3' are the characteristic points for the spans AB and CD, since there is neither loading nor relative deflexion between the ends of those spans.

The characteristic points m_1' and r_2 on either side of B are joined and the line $m_1'r_2$ cuts the vertical through B at the point k. $m_1'i$ is made equal to r_2k. The known point in the R.M. diagram is A, since the moment at the hinged end A is zero. A is joined to the characteristic point m_1', adjacent to B, and Am_1' is produced to cut Bb (in this case at B). This point is joined to r_2. The known point A is joined to i and the line Ai is produced to cut Br_2 at S. S is then a known point on the R.M. diagram in span BC. The point S′ may be found similarly, or, in this case, by symmetry. SS′ produced both ways cuts the verticals through B and C at b and c respectively. The complete R.M. diagram is AbcD. Superimposing that figure as baseline on ABZCD, which is the diagram for the freely-supported spans, gives the complete diagram for the portal, as shown in *Figs.* 2 (*b*). The vertical support-reactions are each 2·5 tons, and the horizontal reactions are each $\frac{Bb}{12} = 0{\cdot}23$ ton acting inwards. The moments at B and C are each 2·78 ton-feet, and the bending moment at the centre of BC is — 7·22 ton-feet.

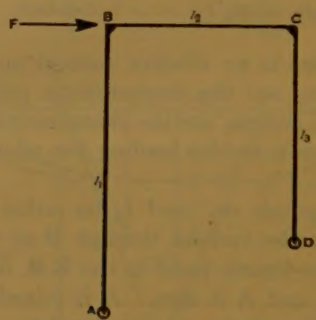
Figs. 2 (*c*) shows that the construction is similar for the case of built-in (fixed) feet, except that the known points in such a case are m_1 and m_3'. In this case, the reactions at A and B are each 2·5 tons vertically upwards, and 0·39 ton inwards. The moments at A and D are each — 1·56 ton-foot.

The constructions given above will not serve for unsymmetrical cases unless the nodes are fixed in position. Before such cases are considered it is necessary to discuss the case of a portal with horizontal loading at a node.

Portal Frame with Horizontal Loading at a Node.

Fig. 3 shows the general case of a portal frame having different values of l, E, and I for each leg. These different values will be indicated by the suffixes 1, 2, and 3 for AB, BC, and CD respectively. It is clear that, since there is no loading that would produce bending moments in the three legs, if freely supported, the heights of the characteristic points in the various sections must be determined from the relative deflexions of the ends of the separate spans. Within the limits of a practical case, B and C will move equal amounts horizontally, but will have no vertical

Fig. 3.



movement relative to one another. If the movement of B and C (say Δ) to the right were known, then it would be possible to obtain the heights of the characteristic points in the various spans, as follows :—

$$\mathrm{m}_1\mathrm{r}_1 = -\frac{2E_1I_1\Delta}{l_1^2} \quad \text{and} \quad \mathrm{m}_1'\mathrm{r}_1' = +\frac{2E_1I_1\Delta}{l_1^2}$$

$$\mathrm{m}_2\mathrm{r}_2 = 0 \quad \text{and} \quad \mathrm{m}_2'\mathrm{r}_2' = 0$$

$$\mathrm{m}_3\mathrm{r}_3 = -\frac{2E_3I_3\Delta}{l_3^2} \quad \text{and} \quad \mathrm{m}_3'\mathrm{r}_3' = +\frac{2E_3I_3\Delta}{l_3^2}$$

Since, however, the value of Δ is not known, it is necessary to make an assumption. It is preferable to assume a value for $\mathrm{m}_1\mathrm{r}_1$ so that :—

$$\mathrm{m}_3\mathrm{r}_3 = \mathrm{m}_1\mathrm{r}_1 \times \frac{E_3I_3l_1^2}{E_1I_1l_3^2},$$

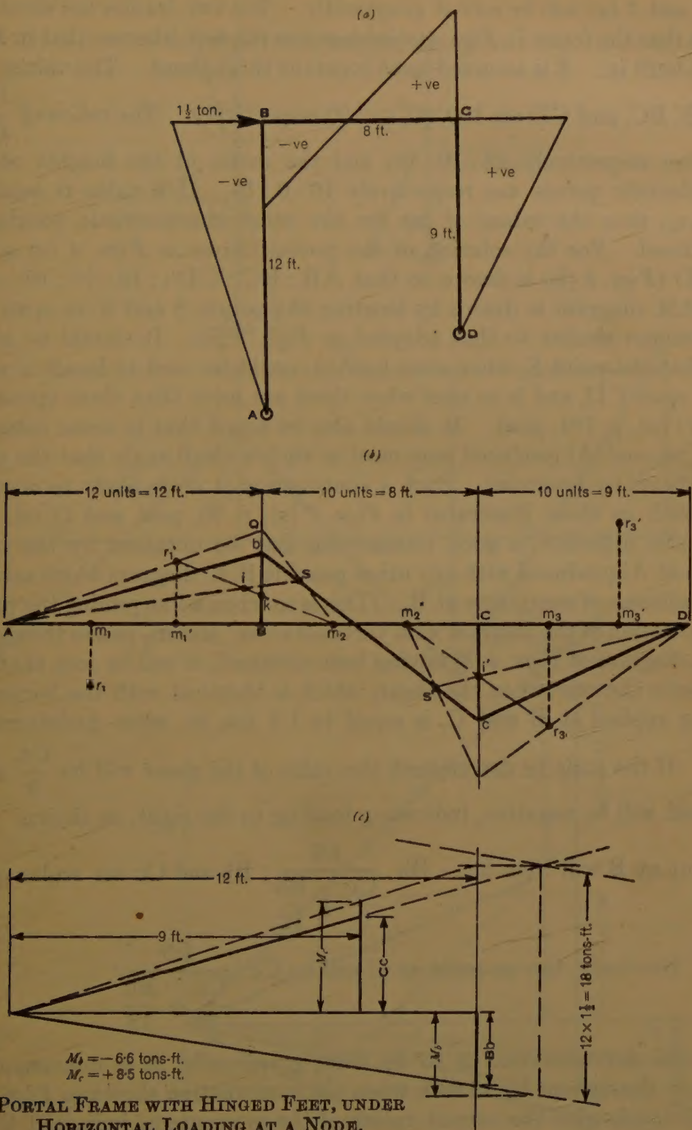
$$\mathrm{m}_1'\mathrm{r}_1' = -\mathrm{m}_1\mathrm{r}_1,$$

and

$$\mathrm{m}_3'\mathrm{r}_3' = -\mathrm{m}_3\mathrm{r}_3.$$

It is now possible to draw the R.M. diagram which corresponds to the assumed deflexion. From this diagram the value of the shear U corre-

Figs. 4.



PORTAL FRAME WITH HINGED FEET, UNDER HORIZONTAL LOADING AT A NODE.

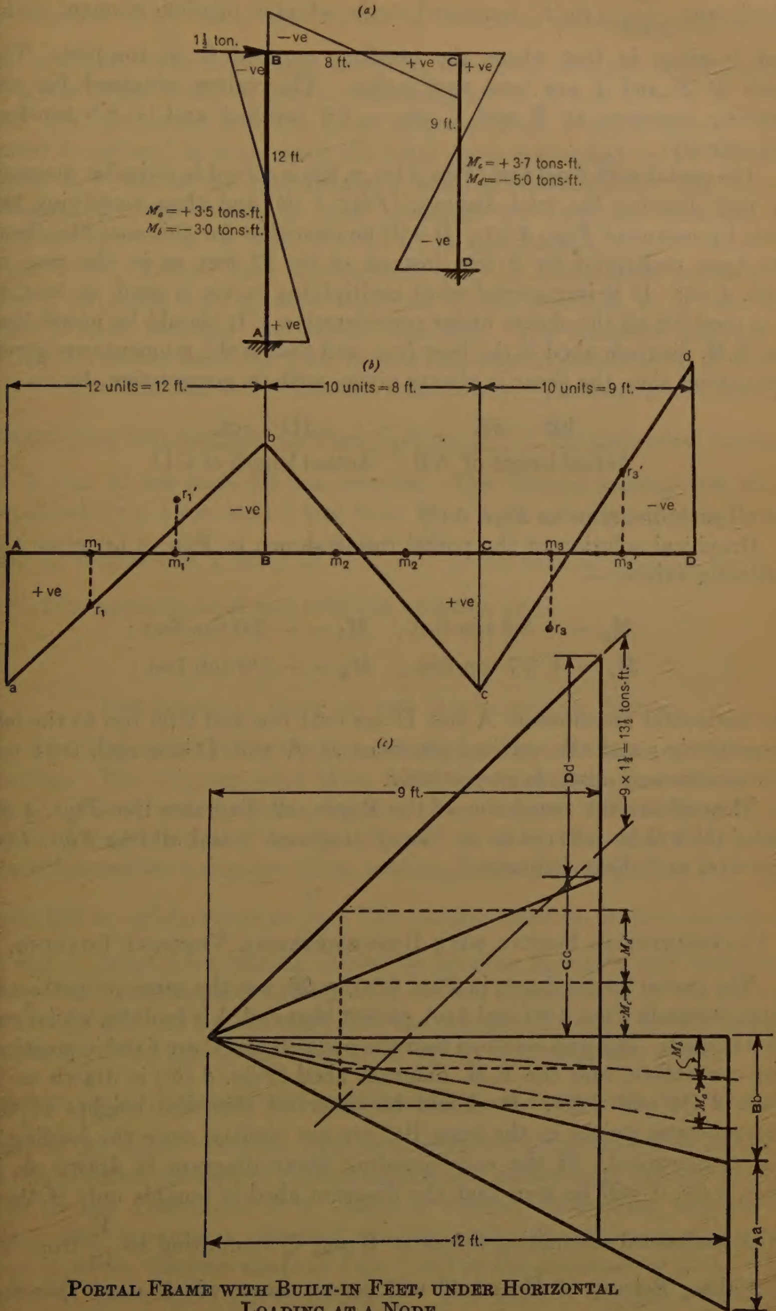
sponding to the assumed deflexion can be found. The correct moments, deflexion, etc., are $\frac{F}{U}$ times those on the arbitrary diagram, and can be found from it, either by calculation or graphically.

To illustrate this procedure more fully, the portals shown in *Figs. 4* (*a*), p. 91, and *5* (*a*) will be solved graphically. The two frames are identical, except that the frame in *Figs. 4* (*a*) is hinged at the feet, whereas that in *Figs. 5* (*a*) is built in. E is assumed to be constant throughout. The values of I for AB, BC, and CD are 100, 80, and 90 respectively. The ratios of $\frac{l}{I}$ are therefore respectively 12 : 10 : 10, and the ratios of the heights of the characteristic points are respectively 10 : 0 : 16. If a value is assumed for m_1r_1, then the values of mr for the other characteristic points are determined. For the solution of the portal shown in *Figs. 4* (*a*) a line ABCD (*Figs. 4* (*b*)) is drawn so that AB : BC : CD : : 12 : 10 : 10. The trial R.M. diagram is drawn by locating the points S and S′ in span BC in a manner similar to that adopted in *Figs. 2* (*b*). It should be noted here that the point S, when once located, could be used to locate a point in the span CD, and is so used when there are more than three spans (see *Figs. 11* (*a*), p. 104, *post*). It should also be noted that in some cases the lines Qm_2 and Ai produced may meet at such a small angle that the point S is difficult to determine. Such a contingency is more likely to occur in cases such as those illustrated in *Figs. 7* (*a*), p. 98, *post*, and *11* (*a*). To avoid this difficulty, a good intersection may be obtained by the intersection of Ai produced with any other possible R.M. diagram which satisfies the conditions of continuity at B. (This is so when a line joining the points of intersection of the diagram with verticals at m_1' and m_2 passes through i.)

The diagram of *Figs. 4* (*b*) having been obtained, it will be seen that this represents the case when the shear, which is identical with the horizontal loading applied at B and C, is equal to 1·5 ton to some undetermined scale. If the scale be determined, the value of the shear will be $\frac{Cc}{9} + \frac{Bb}{12}$ tons and will be negative, indicating loading to the right, as shown. The moment at B will then be $- Bb \cdot \frac{1 \cdot 5}{\frac{Cc}{9} + \frac{Bb}{12}}$; Bb and Cc are scalar quantities. Similarly, the moment at C will be $Cc \cdot \frac{1 \cdot 5}{\frac{Cc}{9} + \frac{Bb}{12}}$.

If the determination is to be done graphically, it is convenient to multiply throughout by 12 feet, when the construction shown in *Figs. 4* (*c*) will obviously give the correct values of the moments as indicated by M_b and M_c, the sign being obtained from *Figs. 4* (*b*) in conjunction with the knowledge that the sway must be to the right, which fixes r_1' above ABCD, or, alternatively, that the shear of the correct diagram must be negative. The magnitude of the sway is $\frac{M_b}{Bb}$ times the assumed value,

Figs. 5.



Portal Frame with Built-in Feet, under Horizontal Loading at a Node.

which was $\frac{12^4}{200E} \times m_1'r_1'$ measured in the adopted bending-moment scale, and is given in feet where the bending moment is in ton-feet. The units of E and I are tons and inches. The values obtained for the bending moments at B and C are $-6{\cdot}6$ ton-feet and $+8{\cdot}5$ ton-feet respectively.

The portal with fixed feet (*Figs. 5* (*a*), p. 93) is solved in a similar manner, by first drawing the trial diagram (*Figs. 5* (*b*)) and then modifying the scale by means of *Figs. 5* (*c*). It will be seen that for this case the shear has been multiplied by 9 feet instead of by 12 feet as in the case of *Figs. 4* (*c*). It is immaterial what multiplying factor is used, so long as it is used for all the shears under consideration. It should be noted that the R.M. diagram abcd is the base-line, and that if the moments are given the correct sign, the shear is always given, with its correct sign, by :—

$$\frac{bB - aA}{\text{Actual length of AB}} + \frac{dD - cC}{\text{Actual length of CD}}$$

for all such diagrams as *Figs. 5* (*b*).

Graphical solution of the portal frame shown in *Figs. 5* (*a*) gives the following values :—

$$M_a = +\,3{\cdot}5 \text{ ton-feet}\,; \quad M_b = -\,3{\cdot}0 \text{ ton-feet}\,;$$
$$M_c = +\,3{\cdot}7 \text{ ton-feet}\,; \quad M_d = -\,5{\cdot}0 \text{ ton-feet}\,;$$

the horizontal reactions at A and D are 0·54 ton and 0·96 ton to the left respectively ; and the vertical reactions at A and D are each 0·84 ton downwards and upwards respectively.

Throughout the remainder of the Paper, all diagrams like *Figs. 4* (*b*) and *5* (*b*) will be referred to as "sway diagrams", and all like *Figs. 4* (*c*) and *5* (*c*) as "shear diagrams."

Unsymmetrical Portal with Unsymmetrical Vertical Loading.

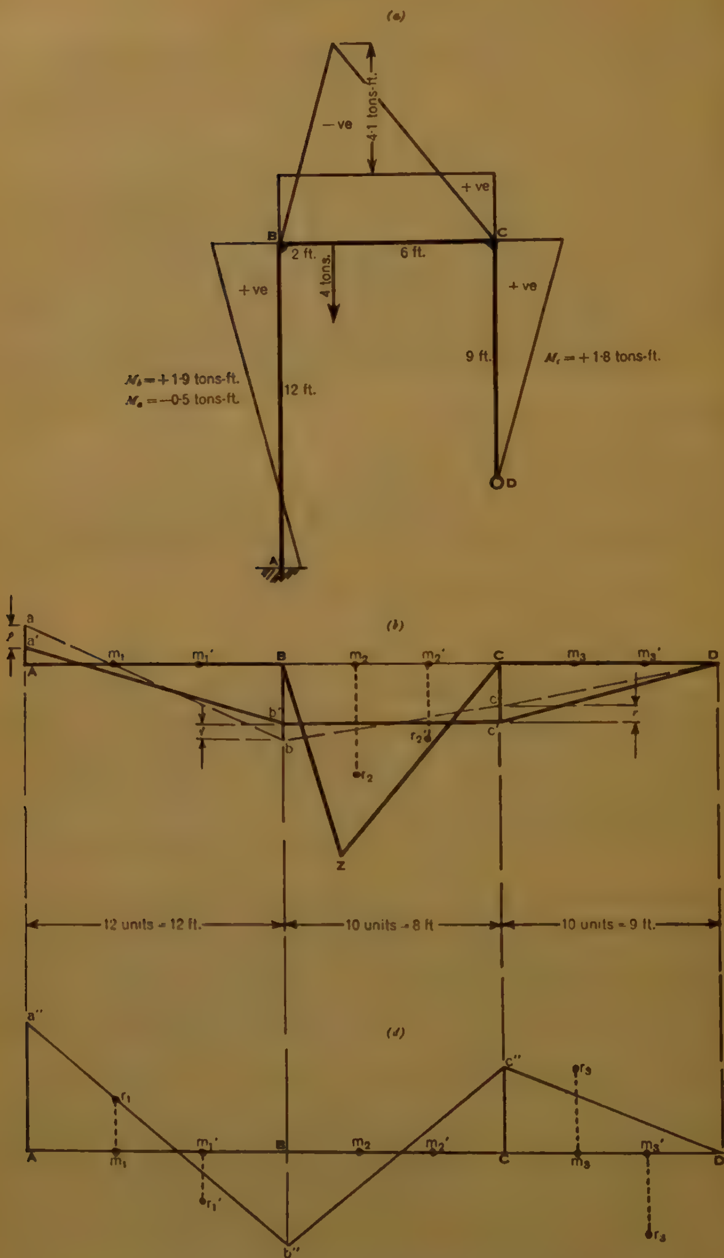
The portal frame shown in *Figs. 6* (*a*), p. 96, has the same properties as those shown in *Figs. 4* (*a*) and *5* (*a*), except that end A is built in, whilst end D is hinged. It is first assumed that the nodes B and C are fixed in position, but can rotate, and the R.M. diagram abcd (*Figs. 6* (*b*)) is drawn as in *Figs. 2* (*b*) and *2* (*c*). It should be observed that the heights of the characteristic points in the span BC are not similar, since the loading is not symmetrical. If the corresponding shear diagram is drawn as in *Figs. 6* (*c*), it will be seen that the diagram abcd is tenable only if there exist horizontal restraining forces at B and C amounting to $\frac{V}{12}$ from left to right. Release of B and C will permit sway, which must therefore

correspond to $\frac{V}{12}$ from right to left to maintain horizontal equilibrium. An arbitrary sway from right to left is, therefore, assumed and the sway diagram (*Figs. 6* (*d*)) is drawn. Moments at A, B, and C of a″A, b″B, and c″C are found and the shear diagram (*Figs. 6* (*e*)) is drawn. The latter is reduced in scale until the total shear multiplied by 12 becomes equal to V, and so the quantities p, q, and r are determined. The correct diagram for the frame is then obtained by applying moments of $+p$ at A, $-q$ at B, and $+r$ at C, in the diagram shown dotted in *Figs. 6* (*b*). This is most conveniently done by shifting the points a, b, and c by equal and opposite amounts to the positions a′, b′, and c′: a′b′c′D then becomes the correct base-line. The complete diagram is ABZCD on the base-line a′b′c′D, and indicates moments at A, B, and C of $-$ 0·5, $+$ 1·9, and $+$ 1·8 ton-foot respectively. The sway is obtained by multiplying that assumed for *Figs. 6* (*d*) by $\frac{\mathrm{p}}{\mathrm{a''A}}$. The horizontal loadings at A and D are each 0·2 ton inwards. The vertical loadings are those produced by a 4-ton load 2 feet from B on a span freely supported at B and C, modified by a shear of $\frac{\mathrm{c'C} - \mathrm{b'B}}{8}$; the vertical loadings are thus 3·01 tons upwards at A and 0·99 ton upwards at D.

The General Case of the Rectangular Portal.

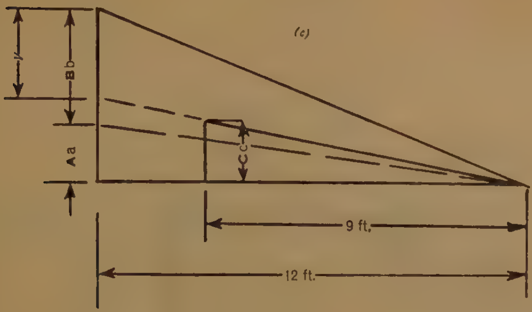
Figs. 7 (*a*), p. 98, shows the frame of *Figs. 6* (*c*) with a more complicated loading. The diagram abcd (*Figs. 7* (*b*)) for fixed nodes is drawn in the usual manner, and the shear diagram (*Figs. 7* (*c*)) is derived from it, all terms being multiplied by 12. It will be seen that the fixed-nodes condition implies a negative shear load applied at B and C (that is to say, from left to right), equal to $\frac{U}{12}$. The loading which is, in fact, applied at B and C, apart from loading caused by bending moments and restraints, comprises 2 tons applied directly at B, 0·5 ton applied at B by the loading in the leg AB, and 1·5 ton applied in the opposite direction by the loading in the leg CD; this totals 1 ton from left to right (negative shear). The amount $\frac{V}{12}$ is, therefore, still wanting and must be applied, by virtue of sway to the right. In *Figs. 7* (*d*) an arbitrary sway to the left is assumed as this reverses the sign of the corrections and makes them directly applicable to the fixed-nodes R.M. diagram without subsequent reversal of sign. The corrections obtained are the scalar quantities p, q, and r, applied to the diagram abcD of *Figs. 7* (*b*)) in the directions of a″A, b″B, and c″C (see *Figs. 7* (*e*)) respectively. The result is the line a′b′c′D of

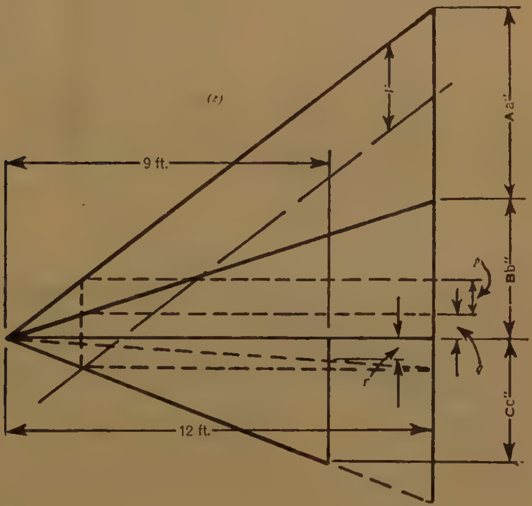
Figs. 6.



UNSYMMETRICAL PORTAL WITH UNSYMMETRICAL VERTICAL LOADING.

Figs. 6.





Figs. 7 (*b*). The complete solution of the portal is as follows :—

The moments at A, B, and C are + 6·02, + 0·38, and + 4·78 ton-feet respectively.

The horizontal load at A is 2·5 + 0·47 = 2·97 tons to the left.

The horizontal load at D is 1·5 − 0·53 = 0·97 ton to the right.

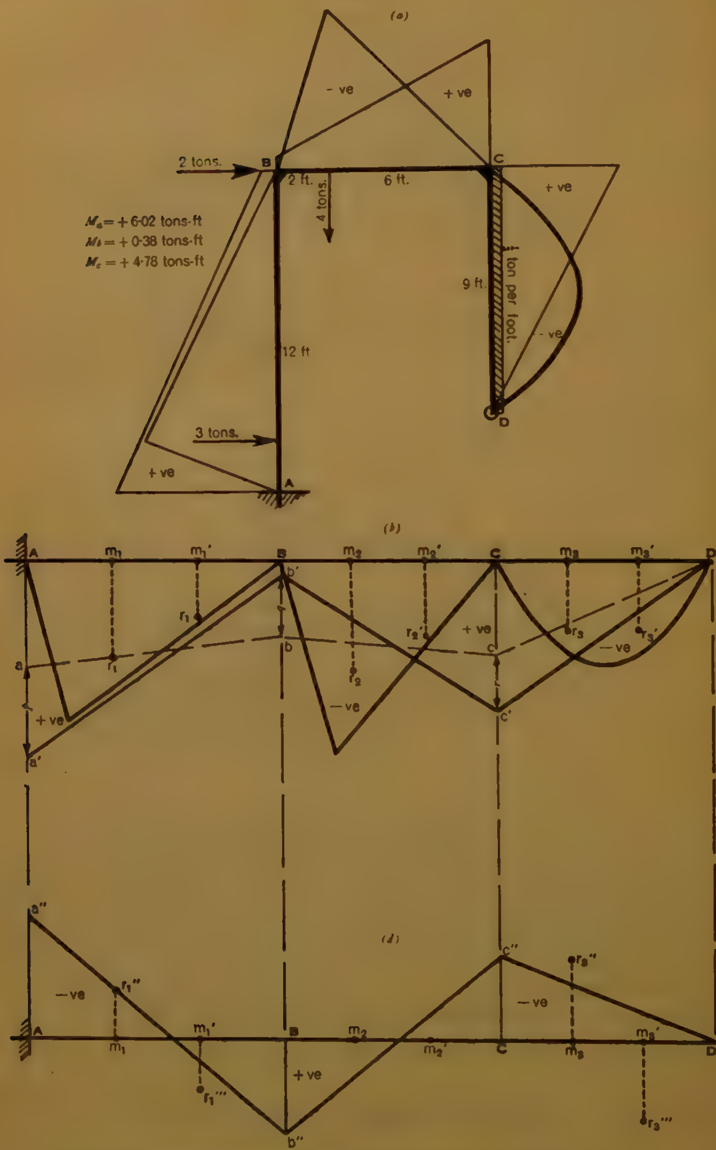
The vertical load at A is 3·0 − 0·55 = 2·45 tons upwards.

The vertical load at D is 1·0 + 0·55 = 1·55 ton upwards.

The sway of B and C to the right is given by :

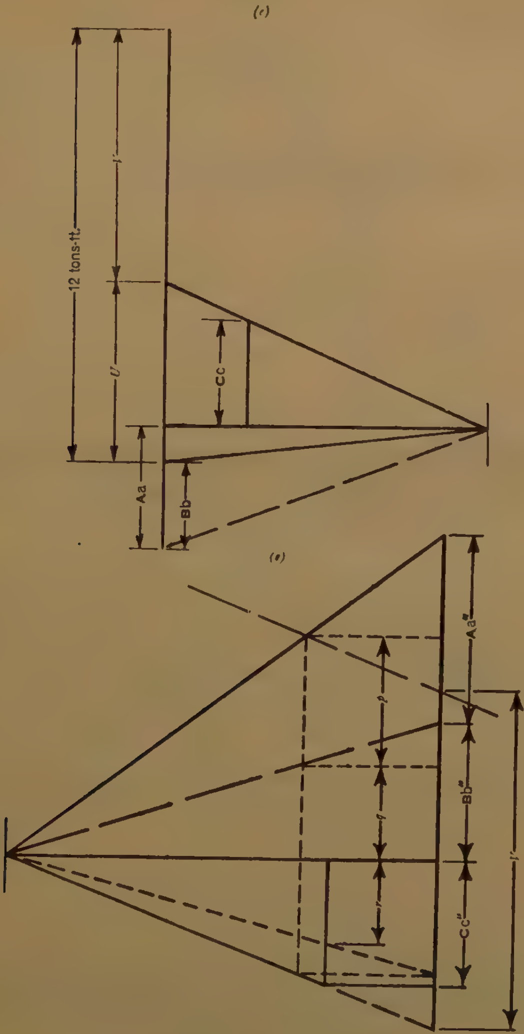
$$\frac{144^2}{200E} \times \frac{p}{Aa''} \times (m_1 r_1'' \text{ in ton-feet}) \text{ feet.}$$

Figs. 7.



UNSYMMETRICAL PORTAL WITH UNSYMMETRICAL VERTICAL AND HORIZONTAL LOADINGS.

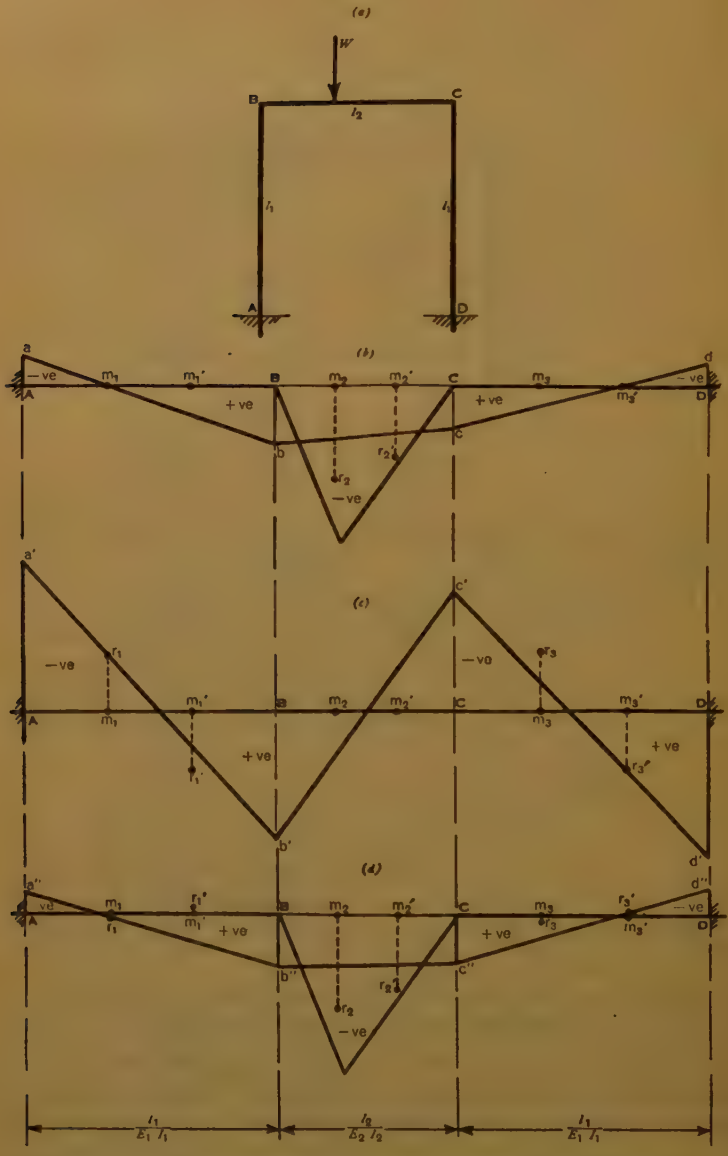
Figs. 7.



Simplified Treatment for a Symmetrical Frame with Unsymmetrical Vertical Loading.

The symmetrical frame in *Figs. 8* (*a*), p. 100, has both feet built in, and is subjected to an unsymmetrical vertical load W. The fixed-nodes diagram is shown in *Figs. 8* (*b*), and the sway diagram for an arbitrary sway to the left in *Figs. 8* (*c*). $\frac{l}{EI}$ for each span is represented by L, with the suffixes 1, 2, and 3 referring to the spans AB, BC, and CD respectively.

Figs. 8.



Symmetrical Portal with Built-in Feet under Unsymmetrical Vertical Loading.

From *Figs. 8* (*b*) :

$$\mathrm{Bb} = -2\mathrm{Aa} \quad \text{and} \quad \mathrm{Cc} = -2\mathrm{Dd},$$

$$l_1 \times \text{shear} = 3(\mathrm{Aa} - \mathrm{Dd}) = 3\frac{L_2}{L_1}\left\{\mathrm{m}_2\mathrm{r}_2 - \mathrm{m}_2'\mathrm{r}_2' - \frac{2}{3}(\mathrm{Aa} - \mathrm{Dd})\right\}.$$

This simplifies to :

$$\text{shear} = \frac{9L_2(\mathrm{m}_2\mathrm{r}_2 - \mathrm{m}_2'\mathrm{r}_2')}{l_1(2L_2 + 3L_1)}.$$

From *Figs. 8* (*c*),

$$\mathrm{m}_1\mathrm{r}_1 = \mathrm{m}_1'\mathrm{r}_1' = \frac{2}{3}(\mathrm{Aa}' - \mathrm{Bb}')$$

and
$$L_1\left\{\mathrm{m}_1'\mathrm{r}_1' - \frac{2}{3}(\mathrm{Bb}' - \mathrm{Aa}')\right\} = \frac{\mathrm{Bb}'}{3} \,.\, L_2.$$

It follows that :

$$\mathrm{Aa}' = \mathrm{Bb}' \,.\, \frac{3L_1 + L_2}{3L_1} \quad \text{and} \quad \mathrm{m}_1\mathrm{r}_1 = \mathrm{Bb}' \,.\, \frac{3L_1 + 2L_2}{9L_1};$$

and
$$\text{shear} = \frac{2}{l_1}(\mathrm{Aa}' + \mathrm{Bb}') = 2\mathrm{Bb}' \,.\, \frac{6L_1 + L_2}{3L_1 l_1}.$$

These two values of the shear have been found from the pure geometry of the two diagrams without regard to sign, and it will be seen that, as both shears are positive, the value of the first obtained requires a minus sign in order that this shall be so. If *Figs. 8* (*c*) is the true sway-diagram, the value of the shear obtained from it added to the value obtained from *Figs. 8* (*b*) should be zero. It follows that :

$$\mathrm{Bb}' = \frac{27L_1L_2(\mathrm{m}_2\mathrm{r}_2 - \mathrm{m}_2'\mathrm{r}_2')}{2(6L_1 + L_2)(3L_1 + 2L_2)},$$

from which :

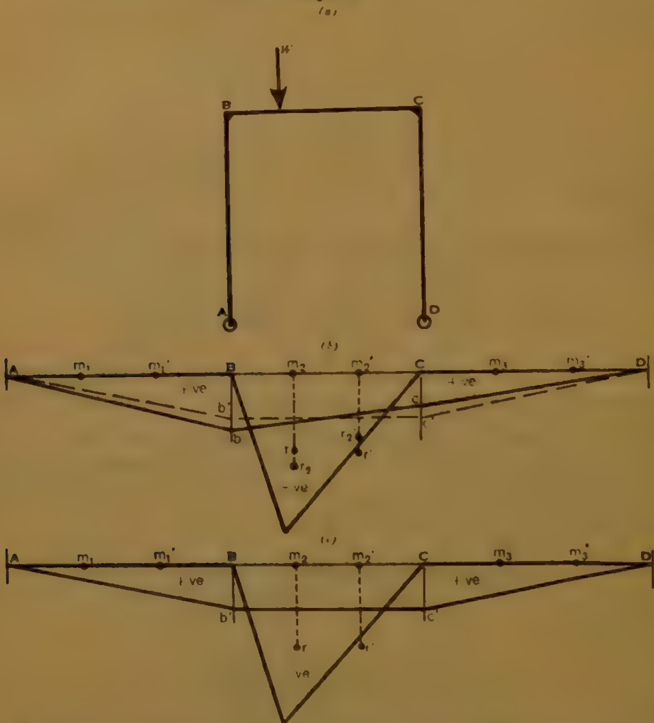
$$\mathrm{m}_1\mathrm{r}_1 = \frac{3L_2}{2} \cdot \frac{\mathrm{m}_2\mathrm{r}_2 - \mathrm{m}_2'\mathrm{r}_2'}{6L_1 + L_2}.$$

It is known also that $\mathrm{m}_1\mathrm{r}_1 = \mathrm{m}_3\mathrm{r}_3 = -\mathrm{m}_1'\mathrm{r}_1' = -\mathrm{m}_3'\mathrm{r}_3'$. It is thus possible to find, from the known heights of the characteristic points in the span BC, the values to be used in the spans AB and CD ; this enables the bending-moment diagram to be drawn at once, as in *Figs. 8* (*d*), with a possible saving of time. A similar method can be evolved for the case of hinged feet, but, for such case, a still simpler method is illustrated in *Figs. 9*, p. 102.

AbcD (*Figs. 9* (*b*)) is the fixed-nodes diagram for the frame of *Figs. 9* (*a*), and Ab′c′D is the true diagram. It is apparent that bc and b′c′ will bisect one another, and so bb′ = cc′. It is clear also that, since the horizontal loads at A and D must be equal and opposite, and no moments exist at these points, then Bb′ = Cc′. b′c′ could therefore be

obtained from bc by drawing a horizontal line through its centre-point. Upon further examination, however, it may be seen that the same object is achieved by adjustment of the characteristic points of span BC, until their heights are equal, the adjustments necessarily being equal and opposite. This means that the adjusted characteristic points must be at the mean height of the free bending-moment diagram. The diagram of *Figs.* 9 (*c*) is thus simply and immediately obtained.

Figs. 9.

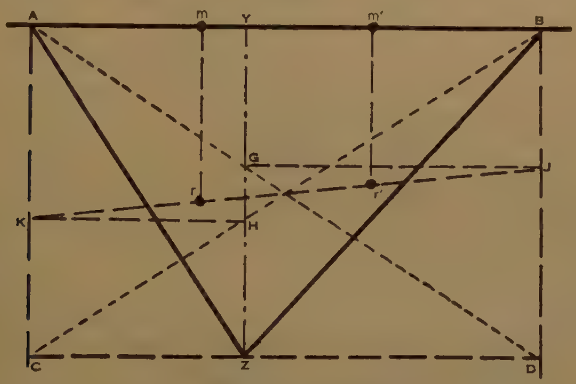


SYMMETRICAL PORTAL WITH HINGED FEET UNDER UNSYMMETRICAL VERTICAL LOADING.

Whilst all the cases treated hitherto in this Paper have dealt with single point-loads, the methods are equally applicable to any system of loading, since it is the heights of the characteristic points and the end reactions which are of primary importance; once these are fixed, the actual shape of the free bending-moment diagram does not affect the R.M. diagram in any way. For the cases of mixed loads the heights of the characteristic points can be obtained for each load separately and added together in the same way as the free bending-moment diagrams. A simple graphical method is available for finding the characteristic points for a single point-load; this is illustrated in *Fig. 10*. The free bending-

moment diagram AZB is drawn, and from Z a perpendicular ZY is drawn to AB. CD is drawn through Z parallel to AB so that AC and BD are perpendicular to AB. A and B are joined to D and C respectively, the lines AD and BC cutting ZY in G and H respectively. From G and H perpendiculars GJ and HK are drawn to BD and AC respectively. JK is then drawn. AB is trisected at m and m′, and perpendiculars mr, m′r′ are drawn to meet JK in r and r′; r and r′ are the characteristic points. The validity of the construction is clear from

Fig. 10.



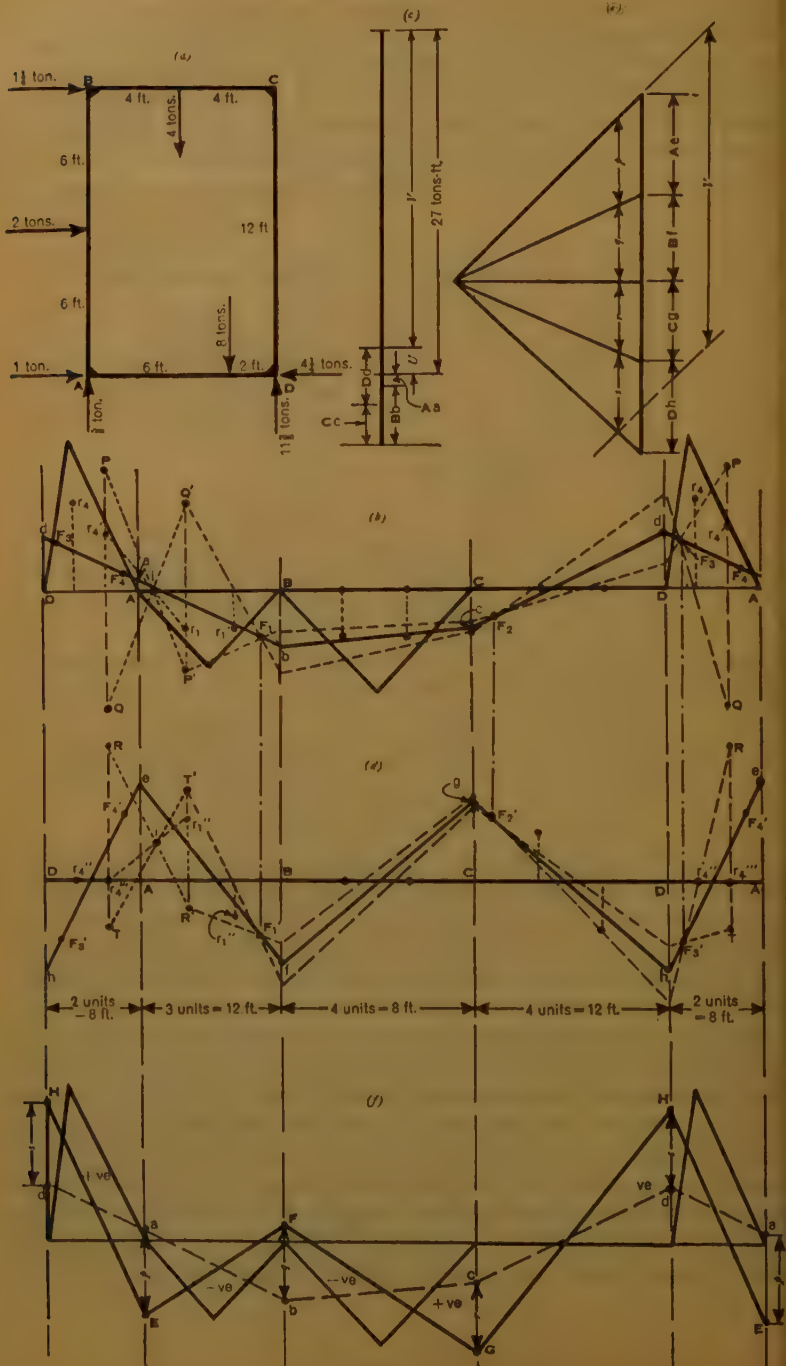
the geometry of *Fig. 10*, for AK and BJ are the respective fixed-end moments at A and B for the given loading,

namely, $$\left(W \,.\, BY^2 \,.\, \frac{AY}{AB^2} \quad \text{and} \quad W \,.\, AY^2 \,.\, \frac{BY}{AB^2}\right).$$

Complete Rectangular Bridge Portal.

In each of the preceding cases, the fixing conditions at A and D were known at the commencement, and so known points in the R.M. diagrams for spans AB and CD from which to commence the construction were established. For the frame ABCDA in *Figs. 11* (*a*), p. 104, however, no such points are known, and there exist, instead, conditions of continuity at the corners A and B. It is possible, in a few special cases, to locate a known point or points from consideration of the symmetry of the loading and of the frame, but, in general, no such points are immediately available. The Hardy Cross method is speedier in operation for this particular case, since both the fixed-nodes moments and the sway moments will usually be found to alter by only very small amounts after the third or fourth balancing; the Author's graphical solution is, however, of academic interest. The method evolved depends upon the consideration that if the nodes are located, but are free to rotate, then there will be a point in at least one span where the existing bending moment will not be altered.

Figs. 11.


(a)
1½ ton.
B
C
4 ft.
4 ft.
4 tons.
6 ft.
2 tons.
12 ft
6 ft.
8 tons.
1 ton.
6 ft.
2 ft.
4½ tons.
A
D
(c)
27 tons-ft.
Dd
Cc
Bb
Aa
Ae
Bf
Cg
Dh
(b)
(d)
2 units = 8 ft.
3 units = 12 ft.
4 units = 8 ft.
4 units = 12 ft.
2 units = 8 ft.
(f)
H
+ ve
– ve
+ ve
ve
E
F
G

This point will be a point of contraflexure in the case where no original moments existed in the frame. It is evident that such a point will be fixed, whatever may be the value of the external moment or rotation applied, so that it can be located by drawing diagrams corresponding to two different rotations. In general, it will be found that fixed points, hereafter known as "flex-points" to distinguish them from points such as S, and the already-known "fixed points" in use in a construction applicable to continuous beams[1], will be located internally in three spans, and externally in the fourth span, although one is sufficient to enable the construction to be carried out.

The fixed-nodes diagram (see *Figs. 11* (*b*)) for the portal shown in *Figs. 11* (*a*) is obtained as follows :—

The frame is opened out in the usual manner, the leg DA being repeated for convenience. Then the characteristic points and the "i"-points are located in each span. It is not necessary at this stage to include the free bending-moment diagrams, although this has been done in *Figs. 11* (*b*). An arbitrary rotation is applied at A. This is obtained by drawing any line such as PiP′ cutting the verticals at the points of trisection of the spans nearest to A in P and P′. P is also located at the other end of the diagram. The dotted arbitrary R.M. diagram beginning at P′ and ending at P can then be drawn by the usual methods. A similar diagram is drawn for another arbitrary rotation, from Q′ to Q. These two diagrams intersect at the points F_1, F_2, and F_3; the production of the lines in BC would locate another point if required. F_1 is used to locate an "S"-point in the span AD by working to the left. This is marked by F_4. There are now two points in the span DA; da is thus located, and, by continuing through the other located fixed points, the whole of the R.M. diagram dabcda is obtained. If it be considered that the sway will take place horizontally, then the shear diagram (*Figs. 11* (*c*)) can be drawn. The same final result would be obtained if a vertical sway were assumed, but the diagrams of *Figs. 11* (*c*), (*d*), and (*e*) would be different. It is seen that the required shear should be 2·25 tons applied to the right at B; this gives the 27 ton-feet in *Figs. 11* (*c*) and determines the correction *V*.

It is now possible to draw the sway diagram (*Figs. 11* (*d*)) by assuming relative deflexions of the ends of the spans AB and CD only. This diagram may be drawn as shown, where the flex-points are located by means of two trial diagrams, or may be drawn by locating them from one trial diagram and verticals from the already-established points in *Figs. 11* (*b*). (These verticals can be located by drawing one diagram for an arbitrary rotation of joint A for an otherwise unloaded frame; the verticals required will pass through the points of zero bending moment. One arbitrary diagram is then all that is necessary to determine the flex-points of *Figs. 11* (*b*)).

---

[1] Ewart S. Andrews, "The Theory and Design of Structures." Chapman and Hall, London, 1934, p. 273.

Figs. 11 (*c*) is obtained from *Figs. 11* (*b*), and the corrections p, q, r, and s are determined. These are shown applied to the fixed-nodes diagram in *Figs. 11* (*f*), where HEFGHE is the final correct R.M. diagram.

In the diagrams shown, the values of I for the spans AB, BC, CD, and DA are 160, 80, 120, and 160 respectively. E is assumed to be constant throughout. The resulting moments at A, B, C, and D are found to be + 6·31, − 1·37, + 8·58, and − 10·76 ton-feet respectively.

Further Extensions of the Method.

The methods detailed above can be adapted, with very little extra work, to the cases where the legs are restrained at one or more points from any movement other than rotary. For instance, if leg CD of *Figs. 7* (*a*), p. 98, *ante*, were pinned at a point E, the span CD of *Figs. 7* (*b*) and (*c*) would require to be divided into two spans CE and ED. Sway characteristic points would be utilized in spans AB and CE only, subsequent work following the same procedure as before.

The further case of a portal with a horizontal strap joining the legs AB and CD can also be dealt with without excessive difficulty, provided the strap is hinged to the legs. A diagram is first obtained assuming the ends of the strap to be fixed in position, thus locating the points of attachment to the legs, but leaving the legs free to rotate. The resulting diagram will determine certain reactions at the ends of the strap which will, in general, be unequal. A further diagram can then be drawn to show the effect of a pure lateral movement of the strap-ends. This will again determine end-reactions at the strap. By using this second diagram to equalize the strap-reactions obtained from the first diagram, the complete correct diagram can be obtained. The work involved is about twice that involved in the solution of *Figs. 7* (*a*), for a frame similarly loaded, and is therefore considerable. When the work and complexity of an alternative solution is considered, however, it is thought that the graphical solution has points in its favour, in spite of the labour involved. As the two cases just mentioned are rare, and require no new principle in their execution, it is not proposed to give them in detail.

The work can also be adapted to cases where there are several straps, as in some special cases of framed buildings. As a lateral movement of one strap would affect the reactions not only at its own ends, but also at all the other straps, it is clear that the work would involve the solution of simultaneous equations, and would lose any superiority due to ease of execution after two or three straps. Here again the method is evident in view of the foregoing.

The Paper is accompanied by two sheets of drawings, from which the Figures in the text have been prepared.

Paper No. 5228.

"An Experimental Investigation of the Propagation of Tides in Parallel and in Convergent Channels.

By Jack Allen, D.Sc., and James Louis Matheson, M.Sc., Assoc. MM. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*) [1]

TABLE OF CONTENTS.

Notation.

Let T denote the tidal period.
,, h ,, the depth of the water.
,, σ ,, $2\pi/T$.
,, c ,, $\sqrt{gh}$.
,, k ,, σ/c.
,, k_1 ,, $\sigma^2 x/hg$.
,, t ,, the time.
,, ϵ ,, an arbitrary time.
,, x ,, the distance to a point measured along the centre-line from the closed end of the inlet.
,, l ,, the length of the inlet.
,, η ,, the tidal rise from datum.
,, η_x ,, the tidal rise at a point distant x from the closed end of the inlet, at time t.
,, η_M ,, the tidal rise at the mouth, at time $t=a \cos(\sigma t+\epsilon)$.
,, a ,, the rise of tide at the mouth when $\cos(\sigma t+\epsilon)=1$; that is, a denotes the height of the high-water surface at the mouth, above datum.

[1] Correspondence on this Paper can be accepted until the 15th April, 1941, and will be published in the Institution Journal for October, 1941.

Introduction.

The periodic rise and fall of the tides ranks as one of the most interesting and important natural phenomena which come under the observation of engineers concerned with navigation, land drainage, and the possibility of utilizing the tides for power purposes. An outstanding feature of tidal motion in coastal waters is the well-known increase of amplitude in sea-inlets, a notable example being that of the Severn, wherein the spring tidal range increases from about 27 feet in Barnstaple Bay to 40 feet at Avonmouth. Mathematical analysis, indeed, shows that such a magnification of amplitude is to be expected, but there appear to be few, if any, experimental data of a quantitative nature to test directly the degree of accuracy of such theory, or to indicate how far the simplifying assumptions made in the process of the analysis render it still applicable in practice.

The late Sir Horace Lamb discussed [1] the change of tidal amplitude in both parallel and convergent channels. In parallel channels with horizontal beds, the theory leads to the result

$$\eta_x = \frac{a \cos kx}{\cos kl} \cos (\sigma t + \epsilon), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

when the rise at the mouth of the channel is assumed to be simple harmonic, that is,

$$\eta_M = a \cos (\sigma t + \epsilon) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1\text{A})$$

On the other hand, for a flat-bottomed channel with convergent sides, the analysis gives

$$\eta_x = \frac{a\, J_0(kx)}{J_0(kl)} \cos (\sigma t + \epsilon) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where J_0 denotes a Bessel function of zero order.

Certain aspects of these theories call for examination. These are :—

(1) How far are the results limited in application by the simplifying assumption that the tidal amplitude is small in comparison with the depth of water in which the wave travels ?

---

[1] "Hydrodynamics" (Cambridge University Press), 4th ed. (1916), pp. 258 and 267.

(2) What interpretation is to be placed upon h, the depth of water? Is it the depth at low, mean, or high tide?

(3) Is the amplification of the tide in a convergent channel unaffected by the degree of convergence, as is suggested by equation (2)?

A further point of interest is the magnitude of the difference between the tide at the top of a channel which is parallel and that at the top of one which is convergent.

Now, with the origin at the upstream end of the channel, equations (1), (1A), and (2), yield the following expressions:

(a) for a parallel channel, $\eta_{(x=0)} = \dfrac{\eta_M}{\cos(kl)}$

(b) for a convergent channel, $\eta_{(x=0)} = \dfrac{\eta_M}{J_0(kl)}$

The significance of these two results is demonstrated by the following Table, which shows the amplitudes obtained by inserting different values of $kl\left(\text{or } \dfrac{\sigma l}{c}\right)$:

TABLE I.

$\dfrac{\sigma l}{c}$	Ratio $\eta_{(x=0)} : \eta_M$		Ratio of results of formulas:— parallel : convergent.
	Parallel.	Convergent.	
0·3	1·05	1·02	1·03
0·5	1·14	1·07	1·07
0·8	1·43	1·18	1·21
1·0	1·85	1·31	1·41
1·2	2·76	1·49	1·85
1·5	14·1	1·95	7·24

It should also be noted that for $\sigma l/c = \dfrac{\pi}{2}, \dfrac{3\pi}{2}$, etc., $\cos(\sigma l/c) = 0$ and $\eta_{(x=0)} = \infty$. Similarly, if $(\sigma l/c) = 2{\cdot}405$, $J_0(kl) = 0$ and $\eta_{(x=0)} = \infty$ in the convergent channel. In practice, however, the analysis is not expected to apply when the amplitude is large in comparison with the depth.

The Table suggests that the magnification is greater in the parallel than in the convergent channel, although the difference is not so pronounced at low as at high values of $\sigma l/c$. In either case, the magnification of amplitude increases with increase in the values of $\sigma l/c$, and therefore anything which increases the effective length of the channel, or anything which increases the natural period of vibration, has the effect of increasing the tide at the upper end of the channel. This result follows since $\sigma l/c$ is directly proportional to the natural period of the inlet. For example,

Lamb [1] deduced that a cylindrical obstacle introduced near the centre of a long and narrow rectangular tank has "the effect of virtually increasing the length of the tank." The importance of this was not realized by engineers until Professor A. H. Gibson, M. Inst. C.E.[2], showed that it accounts for certain unexpected changes in tidal phenomena due to the erection of obstructions in the form of bridge-piers.

Professor G. I. Taylor [3] extended Lamb's analysis to the case of a channel in which both breadth and depth vary and are proportional to the distance from the top. He derived the equation,

$$\eta = KJ_1\{2\sqrt{(k_1x)}\}/\sqrt{(k_1x)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where K is a constant; k_1 denotes σ^2x/hg; h denotes the mean depth at distance x from the top of the channel; and J_1 is a Bessel function of the first order.

He demonstrated that the tidal amplitudes derived from this equation agree closely with observed phenomena in the Bristol Channel, over a distance of about 61·7 nautical miles with the apex at Portishead. For the purpose of this calculation, he regarded the effective depth as that at low water; and the agreement with observation is especially remarkable in view of the wide tidal range involved. Thus, at the entrance to this channel, the range is 27 feet and the low-water depth 120 feet. Again, at Portishead, where the effective (low-water) depth is taken as 0, the calculated range becomes 39·8 feet. It is also interesting to calculate the rise and fall of tide in the Bristol Channel if it be treated as a channel with convergent sides, but with a uniform effective depth equal to (*a*) the average depth at low water over the 61·7 miles concerned, or (*b*) the height of mean tide-level at the seaward end measured above the average level of the bottom of the channel. Utilizing the data plotted in *Fig. 2* of Professor Taylor's Paper, the depth corresponding with (*a*) is 57 feet, whilst that for (*b*) is 73·5 feet. Adopting the formula,

$$\eta_x = \frac{aJ_0(kx)}{J_0(kl)} \cos(\sigma t + \epsilon) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

the comparative data given in Table II are derived.

The methods of adapting formula (2) to this example in the way described above are thus seen to yield a reasonable approximation at

---

[1] *Loc. cit.*, pp. 433–4.

[2] "An Experimental Investigation of the Effect of Bridge-Piers and other Obstructions on the Tidal Levels in an Estuary." Journal Inst. C.E., vol. 8 (1937–38), p. 210 (March 1938).

[3] "Tides in the Bristol Channel," Proceedings Cambridge Philosophical Society, vol. 20 (1921–22), Pt. III, p. 320.

certain places; formula (3), however, is definitely more precise. More recent information for the Bristol Channel[1] indicates that the spring tidal range at Avonmouth (near Portishead) is 40·3 feet when that at Newport is 38·1 feet. This fact brings Professor Taylor's calculated value for Portishead into even closer agreement with nature than appears from his original Table of comparative results.

TABLE II.

Station. *	Distance, x, from Portishead: nautical miles. *	Rise and fall of tide, η: feet.			
		Observed. *	Calculated by:		
			Formula (2).		Formula (3). *
			$h = 57$ feet.	$h = 73{\cdot}5$ feet.	
Section A . . .	61·7	27	27	27	27
Ilfracombe . .	58	27¼	28·5	27·8	27·7
Mumbles . . .	51·6	27¼	31·2	29·7	28·6
Port Talbot . .	47	29	32·8	30·8	29·7
Porthcawl . .	42	28½	34·5	31·9	30·7
Foreland . . .	42	30	34·5	31·9	30·7
Minehead . . .	30	32¼	37·8	34·3	33·1
Watchet . . .	25	34	39·0	35·2	34·2
Bridgwater Bar .	19½	35	40·0	35·9	35·3
Cardiff . . .	15½	36½	40·4	36·2	36·2
Flatholm . . .	15½	37¾	40·4	36·2	36·2
Newport . . .	8	38	41·2	36·8	37·9
Portishead . .	0	42	41·4	36·9	39·8

(* Data in columns marked thus are taken direct from Professor Taylor's Paper.)

APPARATUS USED IN THE EXPERIMENTS.

The Authors' experiments were carried out with the apparatus which is illustrated diagrammatically in *Fig. 1*, p. 112. Three model sea-inlets were used: one with an apex angle of 13 degrees; one with an apex angle of 6 degrees; and one of uniform width 3·5 inches. A few tests were also made with the 13-degree convergent channel blocked at a point 8 inches from its vertex. In all cases the channel-bed was sensibly level. The models were constructed of wood, which was thoroughly soaked with hot paraffin wax to prevent warping; they were contained within a tank of galvanized iron having an enlargement or tide-box at the downstream end, in which tides were generated. Along each side of the tank was a

[1] "Construction and Operation of Severn Tidal Model." Reports by Professor A. H. Gibson, para. 205, p. 52. H.M. Stationery Office, 63–78–2, 1933.

steel bar serving to support movable cross-bars, on which were mounted measuring gauges of the micrometer type, graduated in thousandths of an inch and arranged to take pointers of different lengths as required for the various depths of water. Two sets of pointers were used; one was of normal type, which could be brought down on to the water-surface,

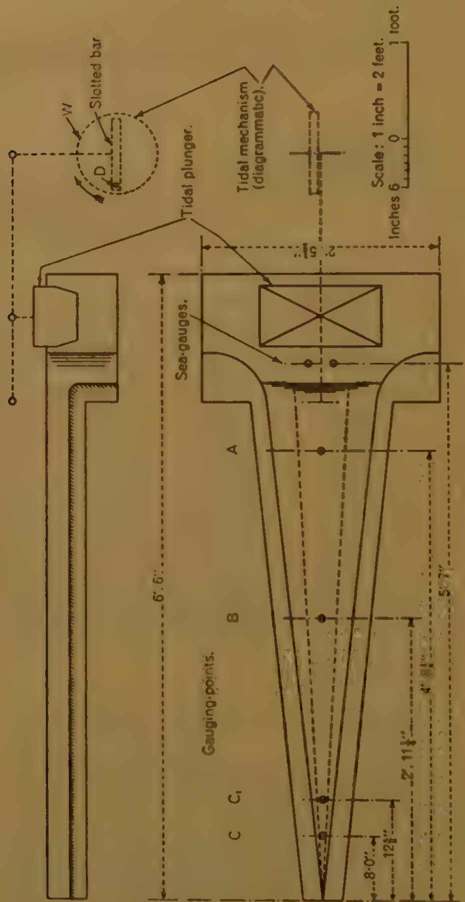


Fig. 1.

MODEL-APPARATUS.

whilst the other was of the hook type, which broke the surface from below. The normal type was used for the measurement of levels during the flood tide, and in particular for the high-water level, whilst the hook type was used for the ebb tide, and in particular for the determination of low-water level. By comparison with a standard pointer-gauge installed in the tide-box, all the different observations were rendered convertible to

depths of water above the inlet-floor, the level of which, relative to the standard gauge, was checked from time to time by bringing the pointer-gauges into contact with a brass cube of known depth resting upon the bed.

For the production of tides a plunger was used, as indicated in *Fig. 1.* This was given a very close approximation to simple harmonic motion by means of a "donkey-pump" mechanism and a linkage. The wheel W carrying the driving-pin D was belt-driven from a constant-speed motor. A pulley on the shaft of this motor could be readily changed to provide a different tidal period. It was also possible to place the driving-pin at different radii on the wheel W, thus enabling three tidal ranges to be generated, which are described for convenience as "spring", "mean", and "neap". Tide-levels were observed at three points in the centre-line of the inlet, and for the purpose of timing the mechanism and the tidal phenomena, a brass strip was fixed to the back of the wheel W. At a certain point in each tidal cycle this strip closed an electrical circuit and produced a flash in a small bulb.

Both convergent inlets were tested with two tidal periods, of 19·61 seconds and 12·16 seconds respectively, whilst the periods during the experiments on the parallel channel were found to be slightly shorter, probably owing to a different adjustment of the belt. The periods for this channel were 19·52 seconds and 12·10 seconds respectively. The tidal ranges, as measured at the seaward gauge A, were as follows:

Table III.—Approximate Tidal Ranges.

	Converging inlet.	Parallel channel.
	inch.	inch.
Neap tides	0·12 to 0·26	0·22 to 0·24
Mean tides	0·27 to 0·49	0·42 to 0·47
Spring tides	0·38 to 0·72	0·58 to 0·60

The temperature varied from 13·1° C. to 18·8° C. No surface tension or viscosity effects have been established.

Experimental Accuracy of the Measurements.

The following conclusions as to the probable accuracy of the measurements are based upon the observed variations of the readings of one observer at different times, and also those of different observers during the same test.

When static water-observations were made for the purpose of connecting the zero readings of the various gauges, a pointer could be adjusted

to $\pm 0{\cdot}001$ inch, whilst in making tidal measurements, the micrometers could be read to $\pm 0{\cdot}002$ inch on the smaller ranges, increasing to $\pm 0{\cdot}005$ inch on the larger, when fluctuations of the surface-level rendered observation more difficult. Two static readings and two tidal observations were necessary, so that the total error in measuring the range at each point was between $\pm 0{\cdot}006$ inch and $\pm 0{\cdot}012$ inch, corresponding with an uncertainty of approximately 5 per cent. of the smallest tides and 1 per cent. of the largest. Similarly the smaller depths may have been in error by 2 per cent., and the greater depths correspondingly less.

Preliminary Tests.

The first experiments were designed to ascertain whether, with the machinery provided, the tide-curve at the entrance to the model was of simple harmonic form. These experiments showed that a very close approximation was obtained, provided that the depth of water was sufficient for the prevention of surging due to the creation and reflexion of waves from the top of the channel and from the plunger. In fact, provided that the conditions (which are discussed later) were favourable to the mathematical theory of the magnification of the tide along the channel, the tide-curve at the entrance was sensibly sinusoidal.

The Main Investigation.

Let ${}_{B}R_{A}$ denote the ratio of the tidal range at B to that at A ;
,, ${}_{C}R_{A}$,, ,, ratio of the tidal range at C to that at A ;
,, ${}_{C_1}R_{A}$,, ,, ratio of the tidal range at C_1 to that at A ;
,, ${}_{L}h_{A}$,, ,, depth of water at A at low water ;
,, ${}_{M}h_{A}$,, ,, depth of water at A at mean-tide level ; and
,, T ,, ,, tidal period, in seconds.

The positions of gauges A, B, C, and C_1 are indicated in *Fig. 1.* The graphs reproduced in Plate 1 are typical of the experimental results.

(i) 6-*degree convergent channel.*

Figs. 2 (*a*) : ${}_{C_1}R_{A}$ plotted against ${}_{M}h_{A}$. T = 12·16 seconds.
,, 2 (*b*) : ${}_{C_1}R_{A}$,, ,, ${}_{L}h_{A}$. T = 19·61 ,,
,, 2 (*c*) : ${}_{C_1}R_{A}$,, ,, ${}_{M}h_{A}$. T = 19·61 ,,

(ii) 13-*degree convergent channel.*

Figs. 3 (*a*) : ${}_{B}R_{A}$ plotted against ${}_{M}h_{A}$. T = 12·16 seconds.
,, 3 (*b*) : ${}_{C}R_{A}$,, ,, ${}_{M}h_{A}$. T = 12·16 ,,
,, 3 (*c*) : ${}_{B}R_{A}$,, ,, ${}_{M}h_{A}$. T = 19·61 ,,
,, 3 (*d*) : ${}_{C}R_{A}$,, ,, ${}_{M}h_{A}$. T = 19·61 ,,

(iii) $3\frac{1}{2}$-*inch parallel channel.*

Figs. 4 (*a*) : ${}_{\mathrm{C}}R_{\mathrm{A}}$ plotted against ${}_{\mathrm{L}}h_{\mathrm{A}}$. T = 12·10 seconds.
„ 4 (*b*) : ${}_{\mathrm{C}}R_{\mathrm{A}}$ „ „ ${}_{\mathrm{M}}h_{\mathrm{A}}$. T = 12·10 „
„ 4 (*c*) : ${}_{\mathrm{B}}R_{\mathrm{A}}$ „ „ ${}_{\mathrm{M}}h_{\mathrm{A}}$. T = 19·52 „
„ 4 (*d*) : ${}_{\mathrm{C}}R_{\mathrm{A}}$ „ „ ${}_{\mathrm{M}}h_{\mathrm{A}}$. T = 19·52 „

Each graph includes the curve of calculated ratios based upon formula (1) or (2). As mentioned on p. 114, the maximum probable error in measuring a tidal range was about ± 5 per cent. for the smallest ranges measured. In general, the observations may be expected to lie within $\pm 2\frac{1}{2}$ per cent., and correspondingly, the ratios of ranges at two places may be expected to be within ± 5 per cent. Curves have therefore been added showing the calculated ratios ± 5 per cent. Points within the band thus formed may be regarded as in agreement with the formulas, although a little extra latitude might be allowed with small tidal ranges and small ratios of amplification.

Discussion of the Experimental Results.

(*a*) On the whole, the results agree better with theory when the mean-tide depth, rather than the low-water depth, is used for h in formulas (1) and (2). Comparison of Figs. 2 (*b*) and (*c*), and Figs. 4 (*a*) and (*b*), Plate 1, illustrates this point. It may be argued, however, that the average low-water depth should be used, and not that at the seaward end of the channel. If that were done, the agreement with theory would suffer still more than by using the low-water depth at A.

(*b*) When the results at gauge C agree with mathematical analysis, so also do those at the intermediate gauge B ; and it may be assumed that the whole channel from the closed end to station A is behaving sensibly as suggested by the theory.

(*c*) Comparison of Figs. 3 (*a*) and (*b*), (*c*) and (*d*), and Figs. 4 (*c*) and (*d*), Plate 1, indicates that the theory breaks down first at the upstream (or closed) end of the channel.

(*d*) Figs. 2 and 3, Plate 1, indicate that, over a wide range of conditions, the convergent law is not affected by the change from 6 degrees to 13 degrees in the apex-angle.

(*e*) It is regarded as established that the amplification of tidal amplitude is greater in a parallel channel than in a convergent channel (for an equal value of $\sigma l/c$).

(*f*) The theory ceases to be valid, in a given channel, if the depth of water is low, the tidal period is short enough, or the amplitude generated at the mouth is sufficiently high. In order to fix rigorously the criterion for the limit of the theory, experiments with many lengths and periods would be required. Dynamical similarity suggests, however, that similar

phenomena should occur if the depth and tidal range at the seaward end were each proportional to the square of the length, and inversely to the square of the tidal period. The experiments show that the formulas are applicable when (*a*) the tidal range at the mouth does not exceed $\frac{1}{4}\,h$; (*b*) the length of the channel does not exceed $1{\cdot}30c/\sigma$, where $c = \sqrt{gh}$, $\sigma = 2\pi/T$; and h denotes the mean-tide depth as measured at the mouth. In the limiting case, when $l = 1{\cdot}30c/\sigma$, the range of tide at the top of a channel having a level bed is 3·7 times that at the mouth, when the channel is of constant breadth, and is 1·6 times that at the mouth, when the sides converge to an apex at the top. Considering the example of the Severn, the mean tide-depth at the mouth is about 130 feet. The tidal range

TABLE IV.

$T = 12{\cdot}16$ seconds.

Range at A: inch.	Mean tide-depth at A: inches.	Ratio of ranges. B : A Calculated.	B : A Measured.	C_1 : A Calculated.	C_1 : A Measured.
0·168	2·406	1·16	1·07	1·26	1·29
0·166	2·406	1·16	1·09	1·26	1·17
0·161	2·401	1·13	1·16	1·26	1·35
0·163	2·405	1·16	1·10	1·26	1·25
0·159	1·672	1·25	1·24	1·41	1·38
0·152	1·418	1·31	1·33	1·51	1·49
0·144	1·167	1·41	1·51	1·67	1·84
0·142	0·962	1·54	1·60	1·91	1·89
0·147	0·707	1·97	1·77	2·64	2·34
0·378	2·277	1·17	1·21	1·28	1·33
0·391	2·071	1·19	1·16	1·31	1·31
0·388	1·821	1·23	1·21	1·37	1·33

there is less than one-quarter of this. Moreover, $1{\cdot}30c/\sigma = 99$ nautical miles, which is far in excess of the actual length of inlet considered. Accordingly the mathematical analysis would be expected to apply, as indeed it does.

(*g*) The effect of obstructing the 13-degree convergent channel by a dam placed 8 inches from its apex was investigated, and the results shown in Tables IV and V were obtained.

In making the calculation for the theoretical results in Tables IV and V, the origin has again been taken as at the apex, as though the channel were not blocked. Within the limits previously indicated, the agreement between measured and calculated values is very close.

(*h*) The mathematical theory, which, incidentally, neglects friction, supposes the times of high water to be identical at all points along the

channel, the length of which is supposed to be limited by the considerations of paragraph (*f*). In most of the experiments the time-differences of high water were small: they did not, in general, exceed 0·5 second, or one-twenty-fourth of the shorter period, and the theory was approximately obeyed. When, however, the depths became low, or the tidal range high, time-differences of as much as 1·5 second were observed, accompanied by surging of the water-surface, and the agreement ceased.

On the other hand, in the Bristol Channel, to which Professor Taylor's analysis refers, the time-intervals between high water at the mouth and at Portishead are considerable. Between Swansea and Avonmouth they are of the order of 1 hour. The critical factor appears to be the preservation of an approximately sinusoidal tide-curve throughout the inlet, and this is usually accompanied by a nearly uniform mean-tide level.

TABLE V.

$T = 19·61$ seconds.

Range at A: inch.	Mean tide-depth at A: inches.	Ration of ranges.			
		B : A		C_1 : A	
		Calculated.	Measured.	Calculated.	Measured.
0·163	2·403	1·05	1·06	1·09	1·09
0·161	2·174	1·06	1·06	1·10	1·10
0·164	1·914	1·07	1·06	1·11	1·12
0·164	1·659	1·08	1·08	1·13	1·16
0·158	1·422	1·10	1·11	1·17	1·22
0·155	1·177	1·12	1·14	1·19	1·25
0·153	0·928	1·16	1·14	1·26	1·24
0·147	0·691	1·23	1·20	1·37	1·39
0·401	2·293	1·06	1·05	1·09	1·11
0·397	2·060	1·07	1·07	1·10	1·10
0·390	1·812	1·08	1·08	1·12	1·13

CONCLUSIONS.

The results of the investigations demonstrate that, within certain indicated limits, close agreement exists between theory and experiment, and they confirm that the amplification is greater in a parallel inlet than in a convergent inlet of equal length and mean depth.

No significant difference was observed in the behaviour of two convergent channels, one of 6 degrees and the other of 13 degrees apex-angle.

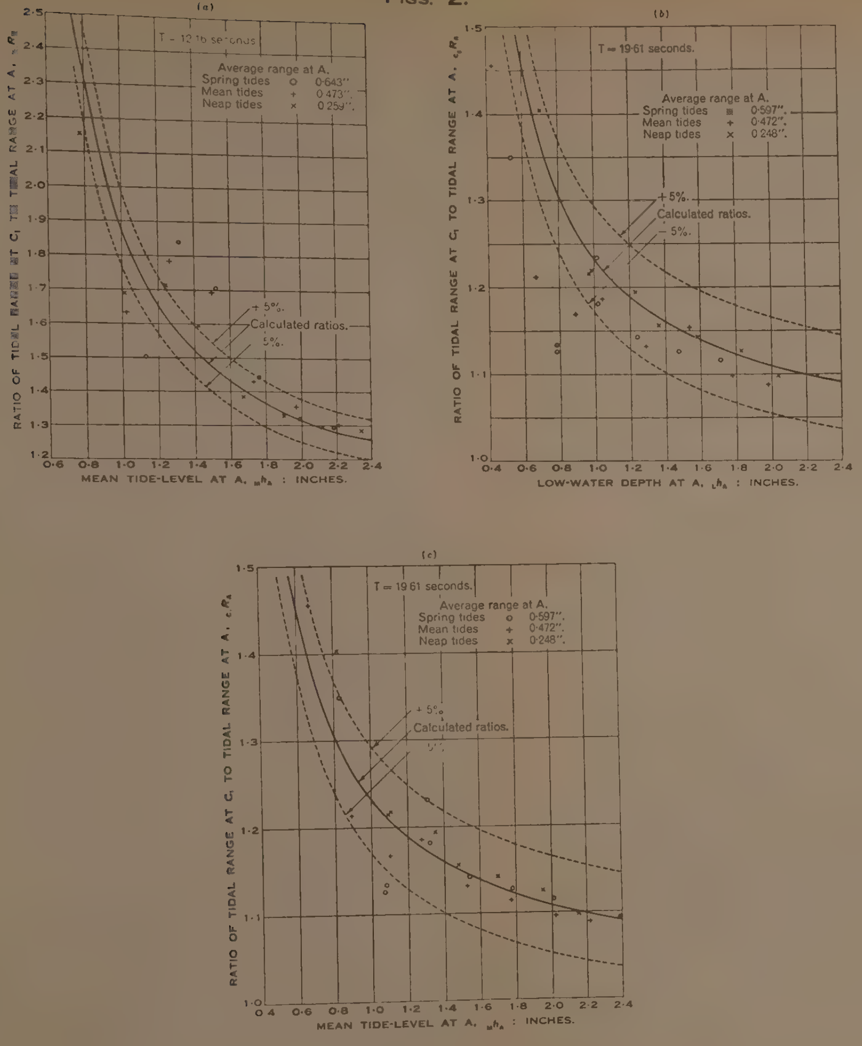
Acknowledgements.

The Authors desire to express their sense of indebtedness to Professor A. H. Gibson, D.Sc., LL.D., M. Inst. C.E., for the facilities placed at their disposal in the Whitworth Engineering Laboratory of Manchester University. They also wish to thank Mr. W. F. G. Crozier, M.Sc., and Mr. K. C. Imrie, M.Sc., research students, for considerable assistance in the experimental work.

The Paper is accompanied by thirteen diagrams, from which Plate 1 and the Figure in the text have been prepared.

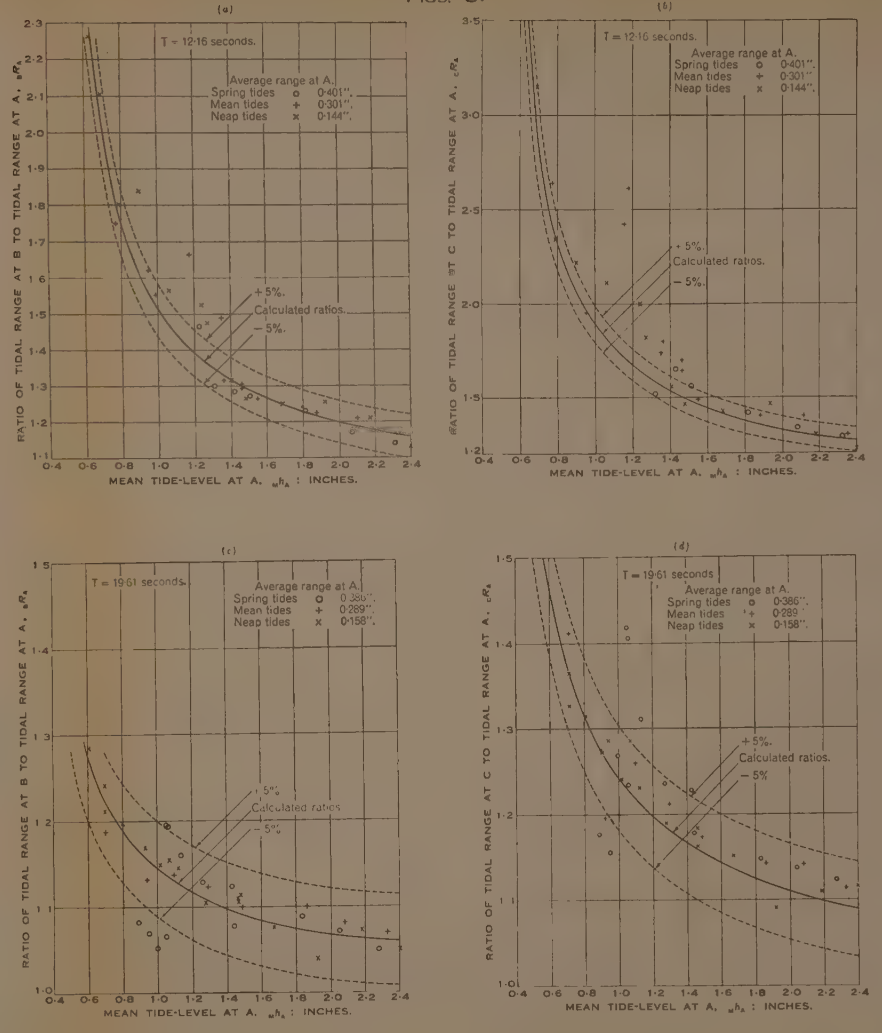
AN EXPERIMENTAL INVESTIGATION OF THE PROPAGATION OF TIDES IN PARALLEL AND CONVERGENT CHANNELS.

FIGS. 2.



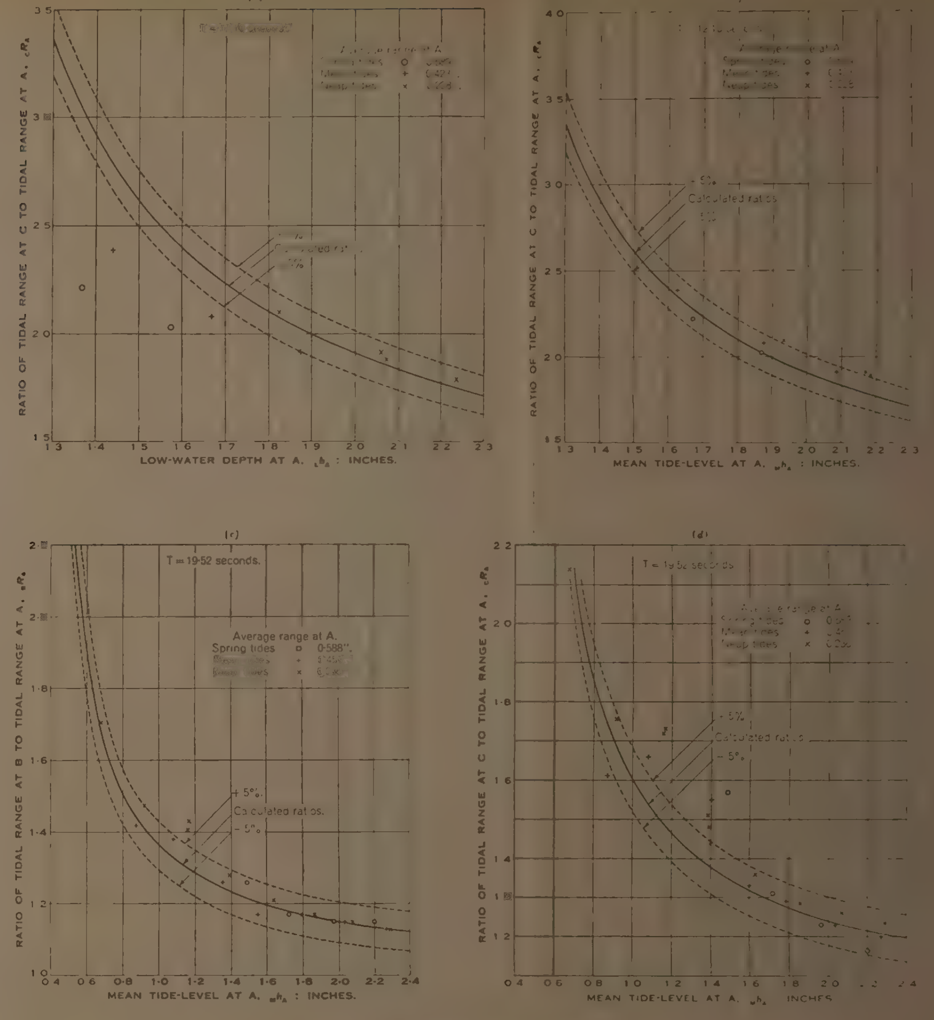
FOR 6-DEGREE CONVERGENT CHANNEL

FIGS. 3.



FOR 13-DEGREE CONVERGENT CHANNEL.

FIGS. 4.



FOR 3-INCH PARALLEL CHANNEL.

CURVES SHOWING THE RELATIONSHIP BETWEEN TIDAL-RANGE RATIOS AND TIDE-LEVELS.

JACK ALLEN and I. L. MATHESON.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

Students' Paper No. 942.

"Foundations for Basement Buildings Adjoining Existing Property."

By SYDNEY KENNETH JORDAN,* Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION.

THE problems associated with the foundations of buildings in congested areas are particularly involved, as it is necessary to ensure not only that the structure is sound after completion, but also that the work is arranged in such a sequence that surrounding properties, as well as highways and public services such as gas and water mains, are not endangered during the progress of the work.

SHORING.

The shoring of adjacent walls and strutting to excavations is a very important part of a Contractor's work, and all timbering should be under the charge of an experienced timberman whose sole responsibility should be the inspection and care of all timberwork, including the slackening and tightening of wedges during wet weather. All proposed shoring and underpinning of party walls should be prepared as early as possible, together with a survey of any cracks in adjoining buildings; agreement should be reached with their owners' surveyors before any work is begun, to ensure that no unjust claims for alleged damage can be made while demolition or rebuilding is in progress.

* Former Stud. Inst. C.E.

[1] Correspondence on this Paper can be accepted until the 15th April 1941, and will be published in the Institution Journal for October 1941.—SEC. INST. C.E.

Fig. 1.

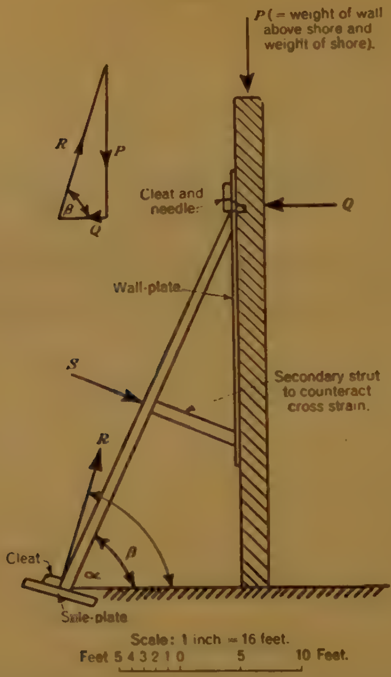
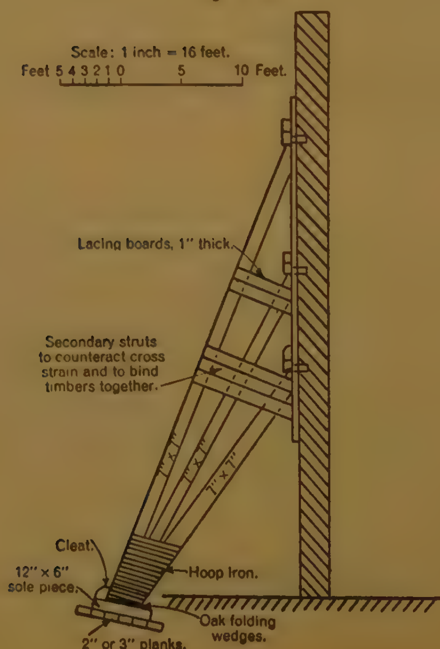

P (= weight of wall above shore and weight of shore).
R
P
β
Q
Cleat and needle.
Q
Wall-plate.
S
Secondary strut to counteract cross strain.
R
β
∝
Cleat
Sole-plate
Scale: 1 inch = 16 feet.
Feet 5 4 3 2 1 0 5 10 Feet.

Fig. 2.

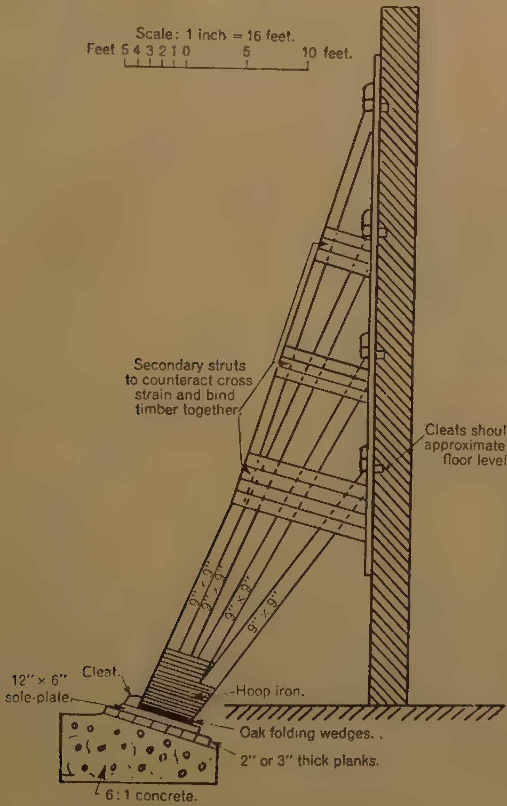

Scale: 1 inch = 16 feet.
Feet 5 4 3 2 1 0 5 10 Feet.
Lacing boards, 1" thick.
Secondary struts to counteract cross strain and to bind timbers together.
7" × 7"
Cleat.
12" × 6" sole piece.
Hoop Iron.
Oak folding wedges.
2" or 3" planks.

Shoring can be divided into three types :—

Raking Shores (*Figs. 1, 2,* and *3*).

Raking shores are, by far, the most common type, but unfortunately have many disadvantages. In the majority of office buildings taken to the maximum height in the city of London, the stanchion foundations adjacent

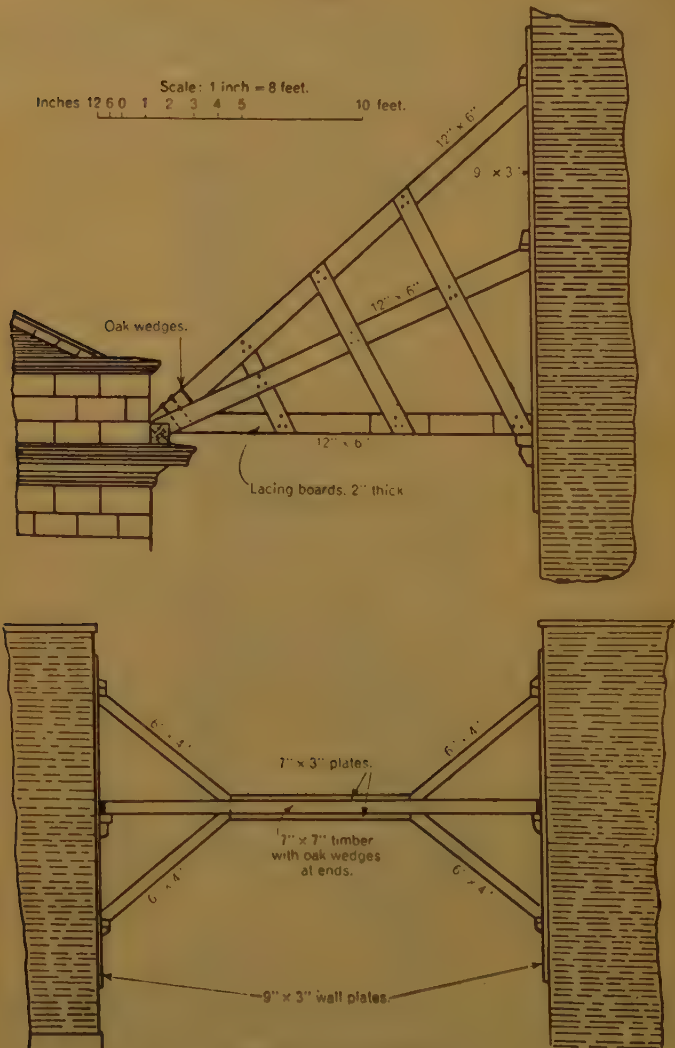
Fig. 3.



to the party walls occupy most of the ground area ; therefore, it is usually difficult to arrange these shores in such positions that they will not interfere with future work or existing chimney breasts. The needles of raking shores should be in such positions that the thrust is taken by the floors of adjoining buildings, but in many cases this is impracticable, as the rakers may foul new steelwork when erected. This should be avoided, as any moving of shores is not only inconvenient, but also dangerous. The sole-plate, bearing on a concrete base on a firm bottom, must be at a lower level

than any subsequent adjacent excavation, and, at all times, care must be taken that rakers are not jarred or disturbed during rebuilding operations.

Figs. 4.



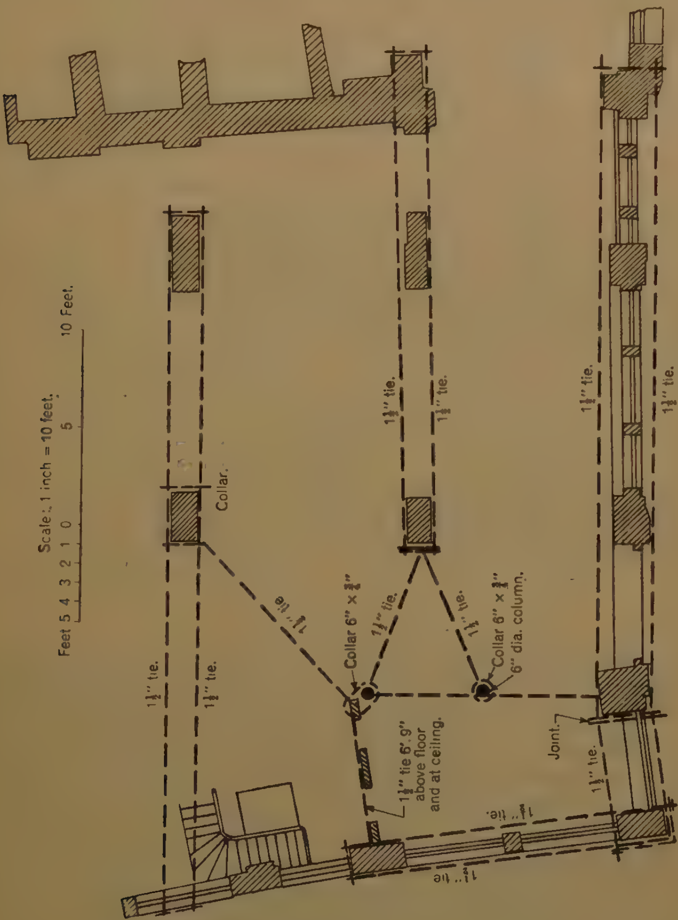
TYPES OF FLYING SHORES.

Flying Shores (*Figs. 4*).

Flying shores should be used in preference to raking shores, as the construction of the foundations, together with the placing of heavy foundation-girders, can be done more conveniently without the congestion of the

three or four timbers and sole-plate at the bottom of raking shores. It is, however, unusual to adopt this type, as in practice there are few cases where the walls to be shored are less than 30 feet apart; for a greater span than this it is undesirable to use a flying shore, as the scarfing and excessive deflexion of the main struts decrease its value as a shore.

Fig. 5.



STEEL STRAPS AND TIE-BARS.

Straps (*Fig. 5*).

Steel straps or channels running horizontally along the wall-face, at ceiling-levels of adjoining property, with steel bars tied back to columns or piers, effectively hold in the wall, and, at the same time do not interfere with the construction of the new work. This method was first introduced by Mr. B. L. Hurst, M. Inst. C.E., and has been used with con-

siderable success in the city of London where other systems would have complicated the work owing to site congestion.

FOUNDATIONS.

Trial holes should be put down as soon as possible, preferably at the corners of the site, thereby approximately determining the slope of the strata, and, if practicable, they should be used as underpinning holes, if underpinning is necessary. Positions of trial holes should be marked on a plan, and accurate records of the strata passed through, and of the standing-water level, should be noted, together with details of existing wall-footings. Trial holes should be carried down to a greater depth than the proposed lowest level, as a poorer stratum may exist below, or, on the contrary, a much better bottom may be found by excavating to a slightly greater depth. The increase in foundation-area, together with the grillage necessitated by a shallow foundation, may be more expensive than the deep excavation with the stanchion load from a steel-slab base spread through the mass concrete at an angle of dispersion of, say, 60 degrees. If practicable, the foundations should be kept above standing-water level to prevent complications entailed by the use of timber runners, steel sheet-piling, or other unforeseen difficulties which often arise when working in water. The usual sequence of foundation construction is :—

(1) Underpinning.
(2) Retaining walls.
(3) Excavation of dumpling and construction of internal foundations.

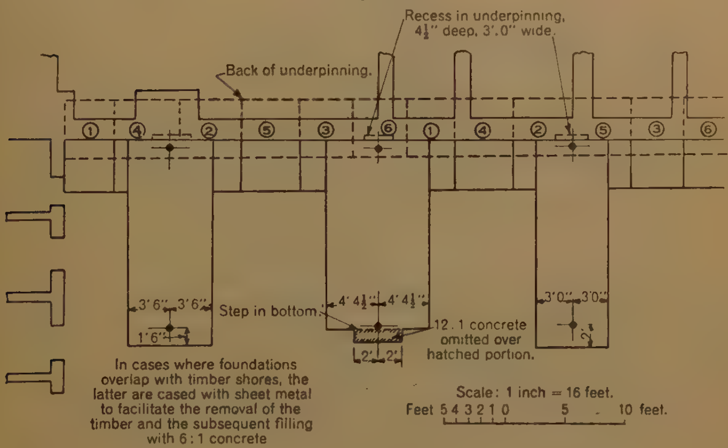
UNDERPINNING.

All walls of adjoining properties should be underpinned if any excavation deeper than the existing wall foundation is to be done within 10 feet of the party-wall face; this, however, is a general rule only, and all proposals for underpinning should be to the satisfaction of the surveyors of adjoining property. Underpinning holes must be excavated from the side of the new work, and, if convenient, should be completed before demolition is begun, and the cross walls removed, as these prevent any possible settlement of the party wall during the underpinning. This is constructed in short lengths under existing footings, and carried down to a lower level than any subsequent adjacent diggings for retaining walls or foundations. Each hole should be from 4 to 5 feet square to allow a skip to remove the spoil, whilst, in order to reduce the area of excavation, the existing projecting footings are cut off as each leg is begun. Underpinning holes should be done in groups of six in the following sequence: 1, 4, 2, 5, 3, 6 (see *Fig. 6*), the legs of the same number being begun simultaneously. This order leads to simplicity in timbering, as holes for legs Nos. 2 and 3 have four-sided timbering, and excavation for inter-

mediate holes Nos. 4, 5, and 6, can be carried out by using back and front timbering only, and removing side timbers of previous holes while working down. With any other sequence of holes, three-sided timbering has to be used in the later legs.

It is important that the setting out of underpinning legs should be marked on the walls and approved by the engineer before any excavation is begun, as the sequence marked on the drawings may have to be altered to suit unforeseen site-conditions such as openings in walls and brick piers, the latter having to be underpinned in two sections. When the excavation of one hole is completed, the concrete work is immediately begun, and, at

Fig. 6.

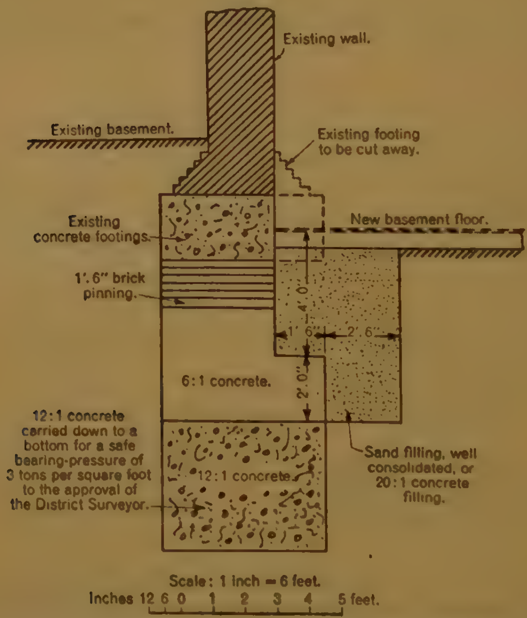


the same time, the excavation for the next-numbered hole is commenced. With this arrangement, not more than one-third of the wall is left unsupported and at the same time the various tradesmen, such as excavators, concreters, and bricklayers, are kept almost continuously at work. If the wall is considered to be in a dangerous condition, each leg should be completely pinned up before the excavation for the following hole is commenced.

Generally all external corners should be underpinned first, but, as in all foundation work, each job must be considered according to the particular conditions. Joggle joints should be cast in sides of the legs constructed first. Such joints should be made by side timber walings, the concrete being cast against poling boards; in addition, longitudinal rods cast in reinforced-concrete underpinning should be left projecting from the sides of the first legs to bond into later legs when constructed. In cases where the bottom is below standing-water level or on loose ballast, it is advisable to arrange the levels of the bottoms so that any subsequent excavation immediately alongside is 4 inches higher, to prevent any danger

of undercutting new work. Bottoms of underpinning legs must be wide enough to carry existing party walls, any raisings to suit new buildings, and the horizontal earth-pressure, whilst the thickness of the vertical part of the legs must be in accordance with the clauses of the London Building Act dealing with party-wall heights. If the underpinning is shallow, mass concrete is used, but deep underpinning should be of reinforced concrete and should be designed as a retaining wall. Rapid-

Fig. 7.



hardening cements should not be used in underpinning, as the excessive shrinkage will cause slight settlement.

Mass-Concrete Underpinning.

A typical example of shallow underpinning was encountered on a job at 50/51 South Audley Street, London (*Figs. 6* and *7*). The excavation was carried down 6 feet from the existing basement, and 4 feet out from the wall, but below this depth the width was reduced, as the men were not impeded by the footings. 12-to-1 concrete was used up to the 6-foot level (see *Fig. 7*), and above this, 6-to-1 concrete, allowing for 18 inches of brick pinning up to the underside of the existing footings with hard stocks in 3-to-1 cement mortar. In places where the underside of the footings was rough, the last inch of pinning was done with "damp"-mixed fine concrete, rammed tight. The brick pinning was toothed at the sides

to bond with the adjoining legs, and the working space was filled with 20-to-1 concrete or suitable excavated material, rammed solid.

Reinforced-Concrete Underpinning.

The procedure for reinforced-concrete underpinning is generally the same as for the mass-concrete type, except when the underpinning is done before demolition and skips cannot be used for removing spoil; in such a case, the working space in front of the wall has to be increased to enable the spoil to be thrown on to platforms.

Retaining Walls.

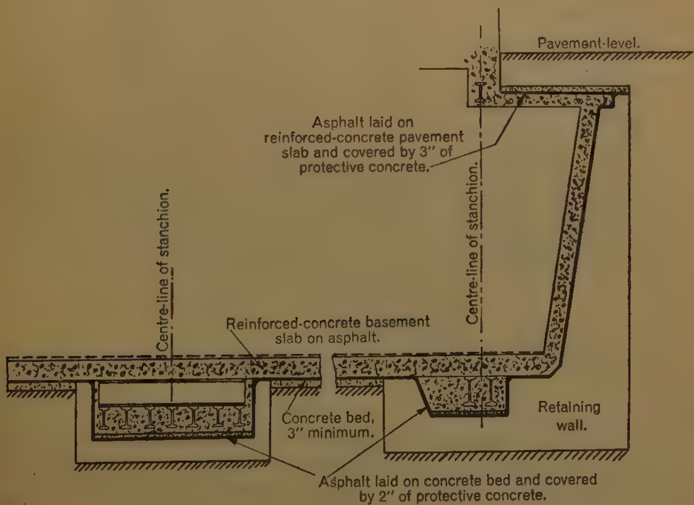
The types of retaining walls most frequently used in steel-framed buildings are :—

(1) Reinforced-concrete cantilever.
(2) Reinforced concrete, strutted.
(3) Mass concrete.
(4) Steel-framed cantilever.
(5) Steel framed, strutted.

Reinforced-Concrete Cantilever Retaining-Wall.

The reinforced-concrete cantilever retaining-wall is the easiest and most convenient type to construct. Its greatest advantage is that, being self-

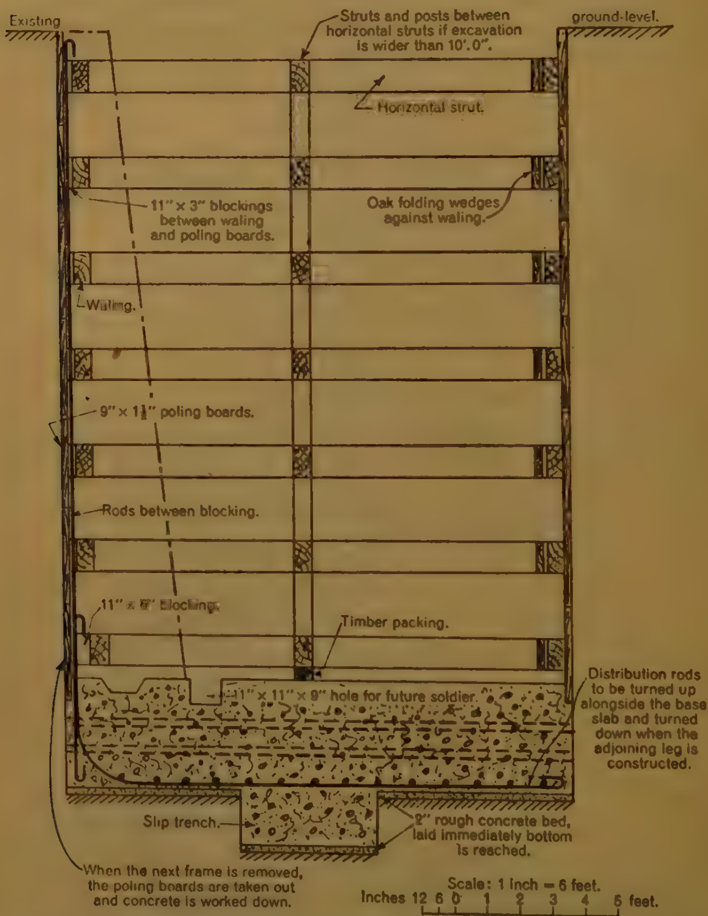
Fig. 8.



supporting, the dumpling can be removed without having to strut the wall; consequently, the construction of internal foundations, fixing of heavy

grillages, and other work in basements is done in an open site, thereby making for quickness and better workmanship. In the majority of cases of retaining walls, they have to serve as foundations for the front row of stanchions, in which case the toe is designed for a temporary condition transmitting the horizontal earth-pressure from the wall on to the ground,

Fig. 9.



and for a final condition, spreading the load from the stanchions in addition.

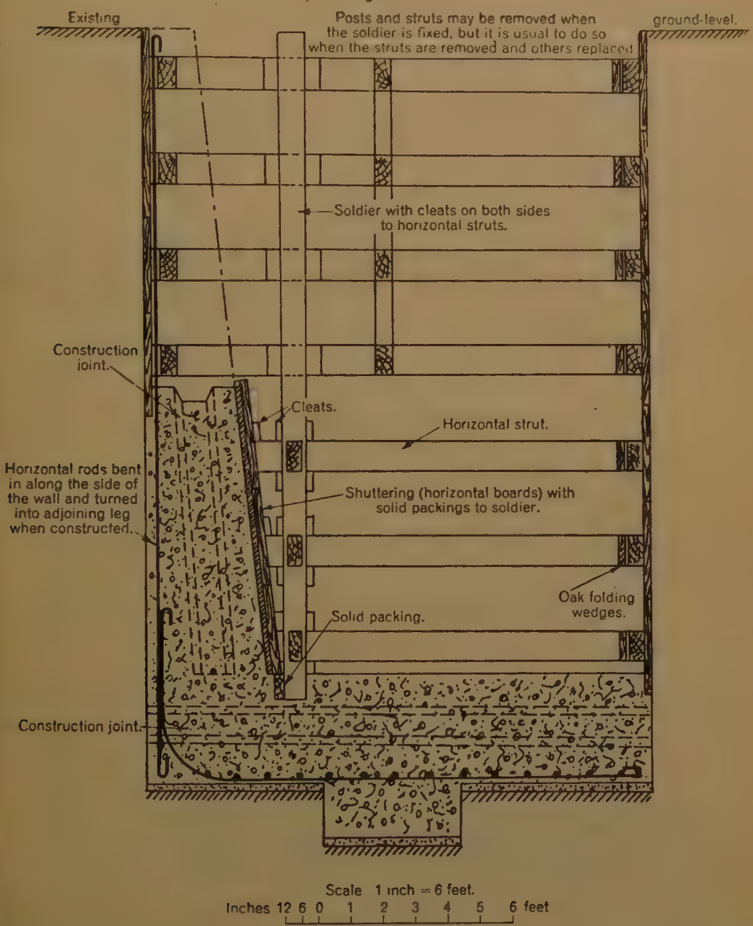
In some cases, when the bottom is good and excavation can be reduced, and it is unnecessary to have a thick reinforced-concrete toe for the temporary condition, the loads from the spreaders are distributed by rolled-steel joists built into the toe.

Fig. 8 shows the construction of a basement tank. Such a construction

is dependent upon the method of waterproofing used. If asphalt or waterproof rendering is used on the front of the wall, the following method is adopted :—

The trench is excavated down to a required bottom, frames being put in as each 3-foot depth is excavated (see *Fig. 9*). The horizontal struts

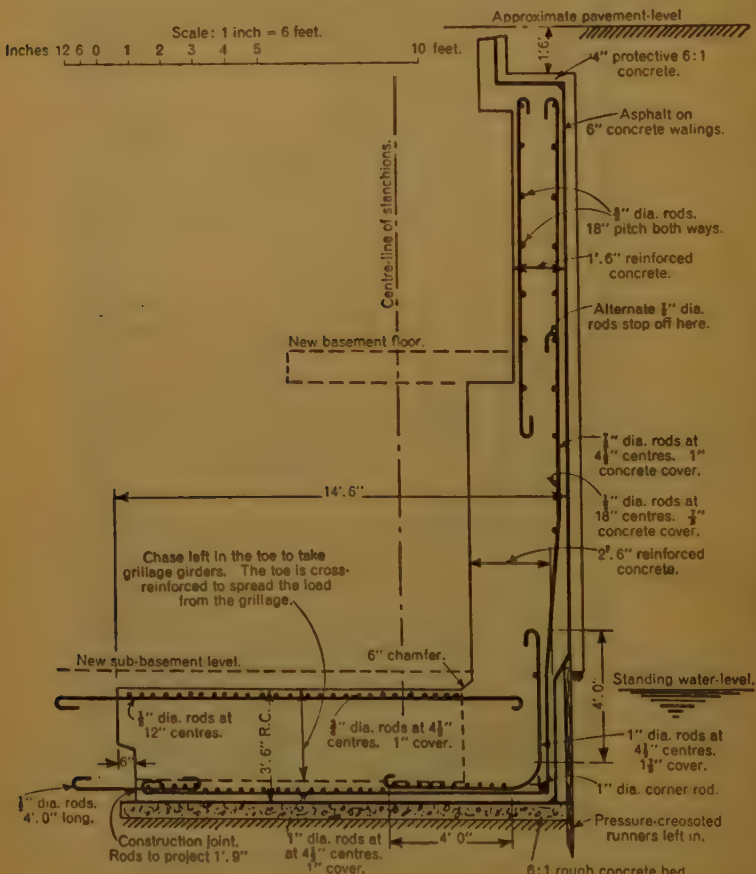
Fig. 10.



are wedged off the back walings, and the front walings are packed off the poling boards by 12-inch by 4-inch blocks, to enable the vertical reinforcement to be dropped from the top, between the walings and poling boards, thereby increasing the effective thickness of the wall and enabling the timber walings to be removed in one piece, and re-used elsewhere; this would be impossible if they were behind the rods.

The lowest 3 feet or so of the excavation is not framed if the ground will stand up, but directly the bottom has been reached and approved, it should be covered with a layer of rough concrete, as it will quickly deteriorate if exposed to the weather. The rods are fixed in position and the toe is completely concreted, horizontal construction-joints being left at the ends, together with holes on top of the toe, one to each set of horizontal struts.

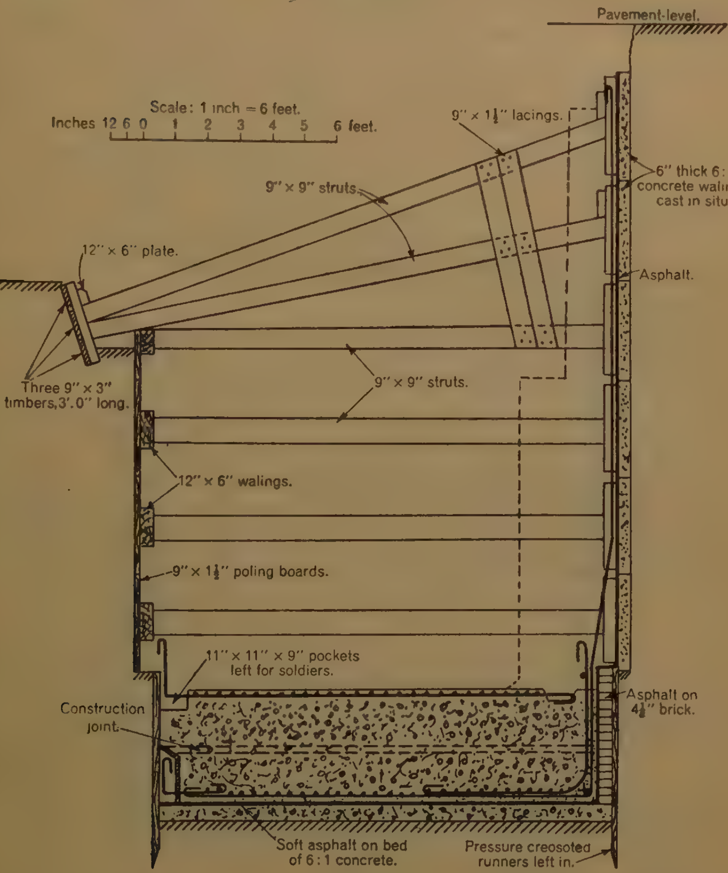
Fig. 11.



Soldiers are then fixed in these holes, and spiked and cleated to the horizontal struts (see *Fig. 10*), except the bottom ones, which are removed with the lowest frame and replaced by struts, one at a time, between the soldier and the back waling. The shuttering for the battered face is fixed to the soldier, and dovetail blocks are fixed at the ends to form construction joints. If the ground will stand vertical, the poling boards are removed

and concrete is rammed back to the earth face; if, on the other hand, it is thought necessary to leave in any timbering, this should have been previously creosoted under pressure. This procedure is repeated for each frame until the wall is complete. The thrust from the ground at the ends of the trench is transmitted through short longitudinal timbers (between the cross-

Fig. 12.

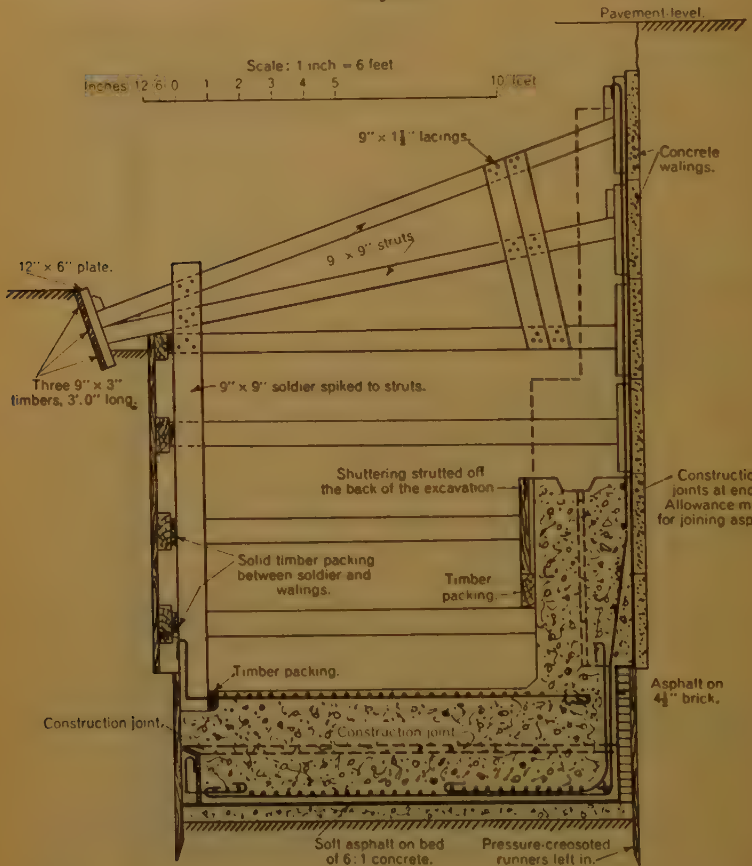


struts) which are transferred on to the soldiers as each set of cross-struts is removed. Longitudinal rods between walings are left projecting, and are turned down to bond into the next length of wall when constructed.

Fig. 11 shows another type of wall (at the National Mutual Assurance Society's premises, Cheapside, London, E.C.), on which the waterproofing has been carried down the back of the wall; this necessitated the construction of a brick or concrete skin as a base for the asphalt. Concrete walings (see *Fig. 12*) were cast in situ at the front of the wall by excavating a 3-foot-

deep trench, driving dowel rods into the ground ahead of the excavation, and casting a 6-inch-thick continuous reinforced-concrete waling which was strutted back to the dumpling. The next 3-foot frame was excavated, the previous operations being repeated and the concrete walings bonded to those above by the rods previously driven into the ground. This

Fig. 13.



procedure was carried out until standing-water level was reached, at which level timber runners were driven down the face of the walings; a 6-inch-thick concrete seat was then laid, and a 4½-inch brick wall was built against runners. Asphalt was laid on the base concrete and up the face of the 4½-inch brickwork and concrete walings. This method was originally devised by Dr. Oscar Faber, M. Inst. C.E.

The subsequent reinforced-concrete construction (see *Fig. 13*) and removal of the timbers was similar to the previous method described.

The former type is preferable, as the asphalt can be laid during the building of the superstructure, and in an open space in one operation, enabling large areas to be completed in sections, thereby reducing the weak points which are likely to occur at asphalt joints. In the latter case, however, no reinforced-concrete lining is necessary to withstand the water-pressure, and so there is a saving of about 9 inches to 12 inches of basement. This saving does not appear to be justified, for considerable difficulty is experienced in asphalting in close-timbered trenches, thus tending to reduce the quality of the work, as well as causing the inconvenience of having to recall the asphalt contractor as each section of retaining wall is ready for waterproofing. On this occasion, however, such a method was the only way practicable, because the site was narrow and had only external columns; it was thus necessary to balance the stanchions across the site and to spread the load longitudinally by a reinforced-concrete raft.

Reinforced-Concrete Strutted Retaining-Wall at Bentall's Store, Kingston-on-Thames.

The reinforced-concrete strutted type of retaining-wall is rarely used, as it is essential to keep the wall strutted until the pavement and the ground-floor slabs, resisting the thrust, are cast; it has, however, the advantage of reducing the thickness, being calculated as a spanning slab between the floor-levels. Care should be taken during calculation to ensure that girders trimming openings in floors, such as stair- and lift-wells, are strong enough about the vertical axis to transfer the thrust to the stanchions.

In this particular instance (*Fig. 14*) the wall was constructed before demolition, the existing brick retaining-wall being underpinned and used as a backing for the asphalt.

Mass-Concrete Retaining-Walls.

All engineers are familiar with the mass-concrete type of retaining-wall, but owing to the thickness required for stability, such walls are rarely used in buildings where floor area is valuable.

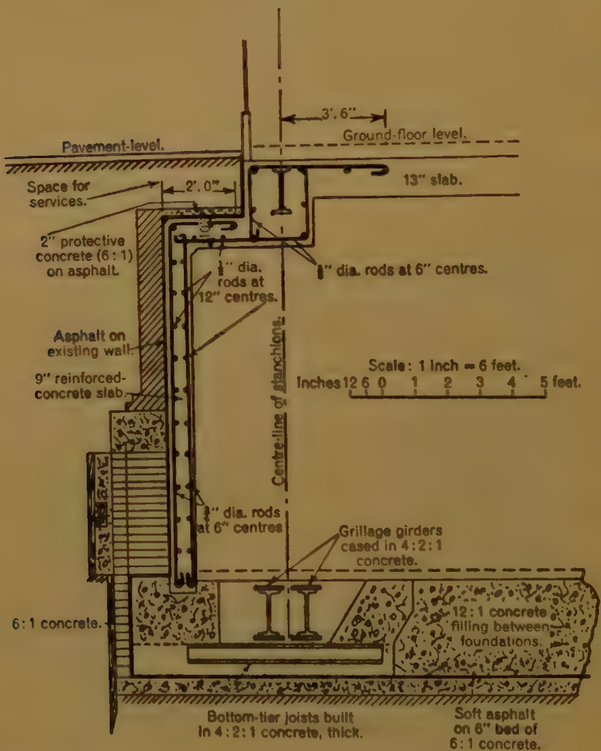
Steel-framed Cantilever Retaining-Wall at Bentall's Store, Kingston-on Thames.

Fig. 15 shows an unusual type of steel-framed cantilever retaining-wall at Bentall's store. In this case it was imperative to reduce the excavation as much as possible, since the bottom was about 5 feet below standing-water level. Consequently the thickness of the toe at the grillage trough was reduced to 1 foot, which would have been impossible in reinforced concrete. The frames were at 3-foot centres, the intermediate spaces being filled with mass concrete. In this case, the joist frames were used to withstand the horizontal earth-pressure in the temporary condition, and are strong enough to spread the stanchion loads finally.

Steel-framed Strutted Retaining-Wall at Peter Jones' Store, Sloane Square, London.

The steel-framed strutted type of retaining-wall (*Fig. 16*) has, of course, the same disadvantages and advantages as the strutted reinforced-concrete wall, but is more practicable for walls of more than one floor height. The excavation is carried out in exactly the same manner as for the cantilever types, except that blocked walings are not necessary.

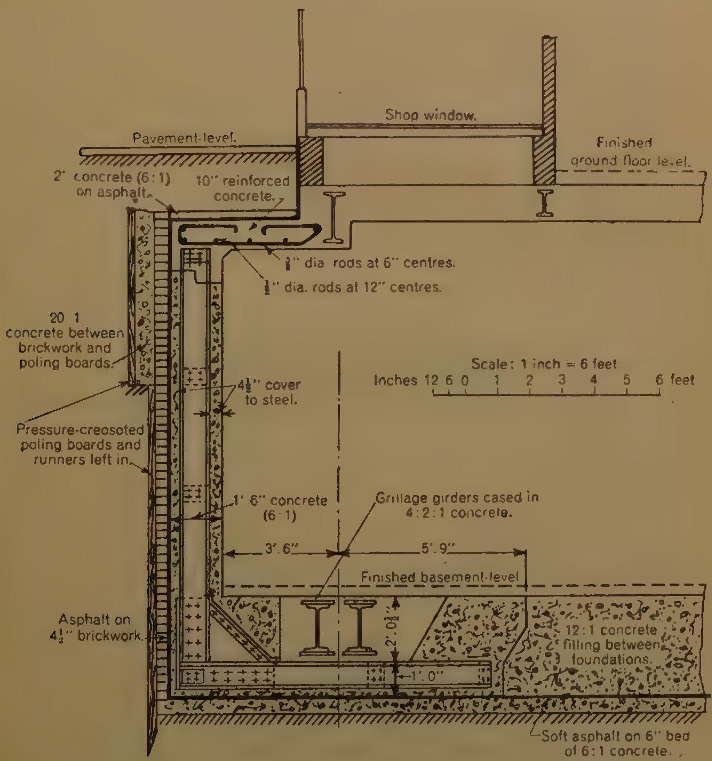
Fig. 14.



Immediately the bottom is reached, a bed of concrete is laid and bond rods are cast in, chases being left in the top. The frames are erected, and the bond rods are bent over the tops of the joists, to prevent the back of the wall lifting. Cleats are riveted to the underside of the joists to fit in the chases, in order to prevent the upper part of the wall from sliding due to horizontal pressure. The backs of the vertical joists are kept 9 inches from the face of the excavation, to suit 2-inch poling-boards and 6-inch-thick walings, thus giving 1 inch clearance between the latter and the joists, and so enabling all the timbering to be withdrawn. Pockets are

left in the wall and toe where struts occur, so that the latter can be removed and used again on another section of the wall, or in the superstructure; but, of course, they are not removed until the basement-floor slab is cast. The vertical joists were designed as cantilevers from the basement-level, to span between the sub-basement and the basement and to take the direct tension induced from the raking struts; the bottom joists are subject to the bending due to the upward pressure, the tension from the

Fig. 15.

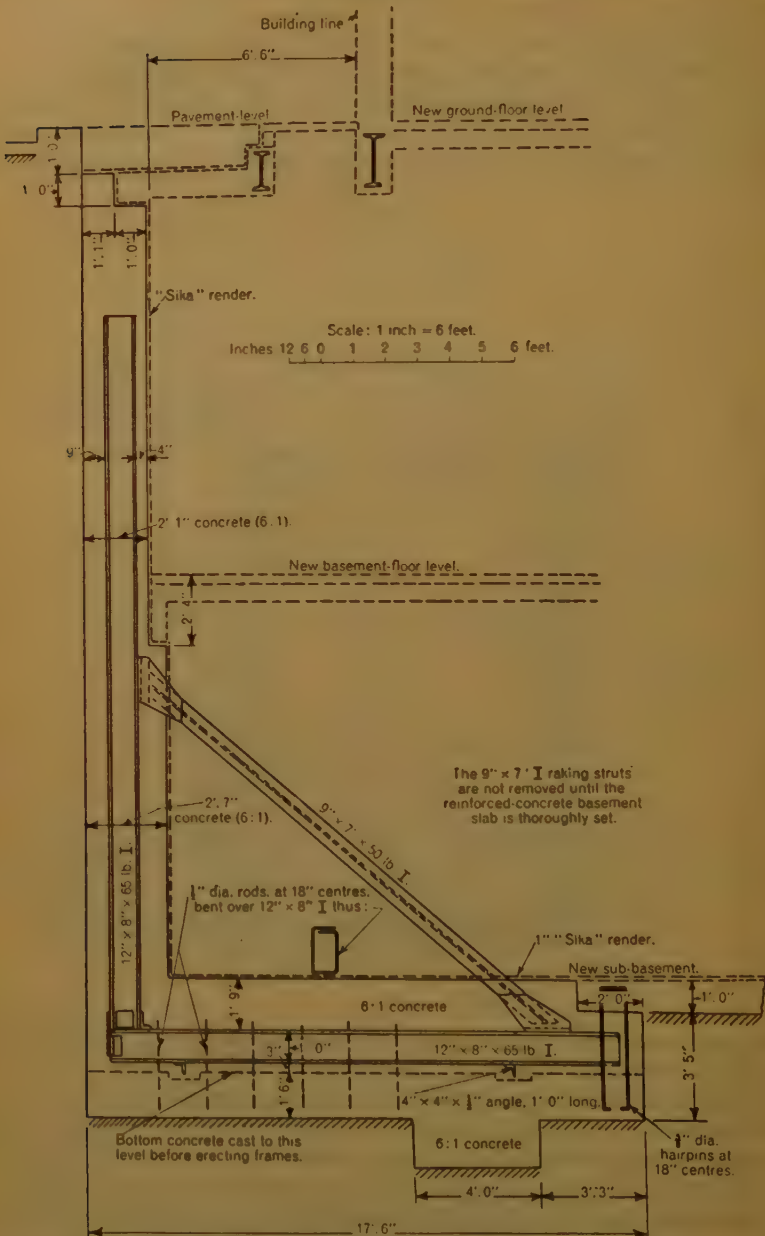


vertical joist acting as a point load at one end, and the downward force from the raking strut acting as a point load at the other end.

At sections of the wall where stanchions occurred the bottom joists were used to spread the load to the full width of the toe, the sizes being modified to suit permanent conditions as necessary.

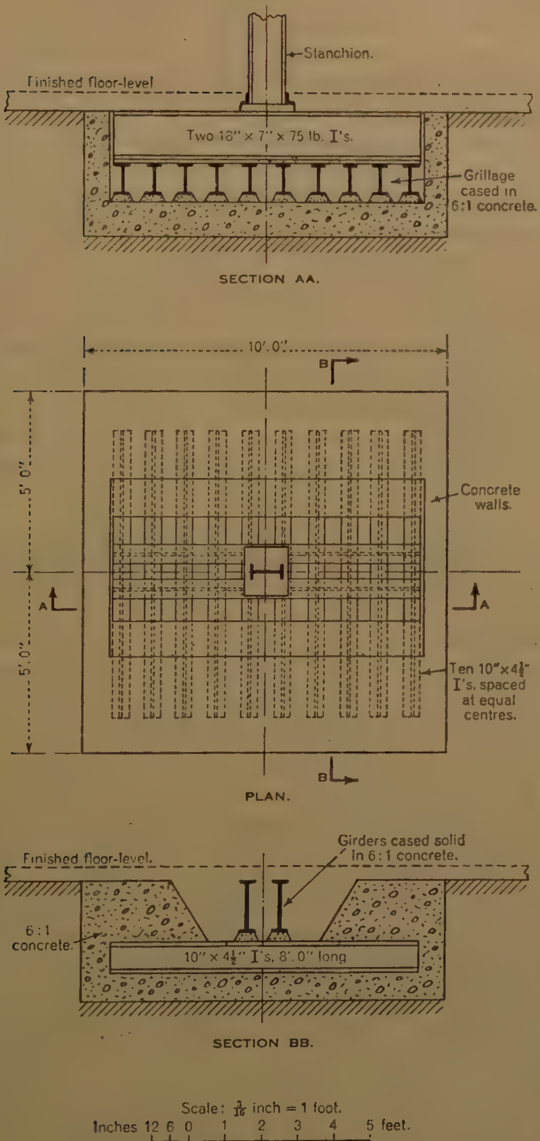
After the underpinnings and retaining walls have been constructed, the dumpling may be excavated, and then the foundations for internal columns can be constructed. The most common type is the grillage foundation (*Figs. 17*), with which all structural engineers are familiar.

Fig. 16.



Steel-Framed Strutted Type of Retaining Wall.

Figs. 17.



Stanchion.
Finished floor-level
Two 18" × 7" × 75 lb. I's.
Grillage cased in 6:1 concrete.
SECTION AA.
10'.0".
B
Concrete walls.
5'.0".
5'.0".
A
A
Ten 10"×4½" I's. spaced at equal centres.
B
PLAN.
Finished floor-level.
Girders cased solid in 6:1 concrete.
6:1 concrete.
10" × 4½" I's, 8'.0" long
SECTION BB.
Scale: 3/16 inch = 1 foot.
Inches 12 6 0 1 2 3 4 5 feet.

The thickness of the concrete underbed is calculated by spreading the load from the bottom-tier joists at an angle of dispersion of 60 degrees, the joists being usually kept 1 foot from the edge of the concrete.

For large foundations it is advisable to have concrete walls to retain earth from falling on to the concrete during the grouting of the grillage, but in the case of small foundations these can be dispensed with by using temporary timbering. The use of such timbering is impracticable for large foundations, as the necessary intermediate struts to long walings hinder the placing of the heavy grillage girders.

In cases where the excavation depth must be a minimum, the bottom-tier joists are built into the foundation, allowing a minimum concrete thickness of 4 inches under joists. In this case, however, care should be taken to ensure that the pressure from the top-tier girders on the grout is not excessive, the width of the flange-plates being fixed to give a stress of not more than 30 tons per square foot on 1 : 2 : 4 quality concrete.

Where columns are external or adjacent to party walls, and where it is impossible to encroach beyond the boundary of the property, it is necessary to balance the loads from these columns with those of an adjacent row of columns. This should be borne in mind when preparing the preliminary steel framing, if architectural considerations permit.

Basement Tanks.

An asphalt lining to basements is the most common method of water-proofing underground structures, but owing to the settlement of foundations after the completion of the building, no matter how small, there is always the risk of the asphalt cracking ; this, however, can be reduced to a minimum by specifying soft asphalt, which is quite plastic. Another fault is the weakness of dirty joints or the possibility of poor workmanship. These faults may, however, be minimized by thoughtful design in arranging for large areas to be laid in positions which do not cramp the workmen in almost inaccessible positions, such as close-timbered trenches. Reinforced-concrete linings have to be provided on the walls to resist horizontal water-pressure (see *Fig. 8*), and the floor must be designed to resist upward hydrostatic pressure. The resultant uplift, over and above the dead weight of the concrete floor-slab, is taken up by spanning the slab on to the concrete foundations, dowel rods to take the tension being built in the foundations as necessary.

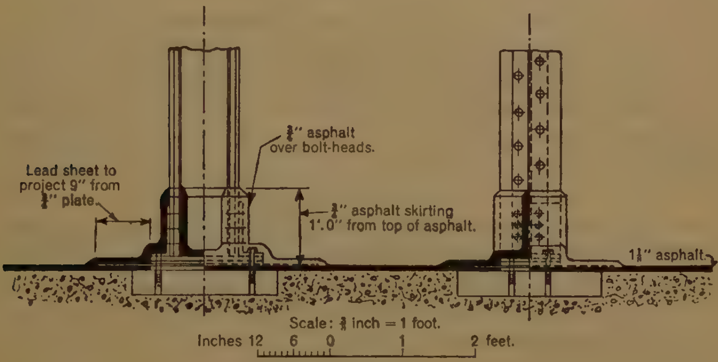
The asphalt can either be carried down behind the retaining wall and under the foundations, or alternatively in front of the wall and under the grillages or over the top-tier girders, in which case, seals are needed at the stanchions. The disadvantages of the former method are that the grillage pits must be asphalted, and a 2-inch protective concrete bed must be laid before commencing the erection of the steelwork. The grillages must

also be designed so that the pressure of the steel on the asphalt does not exceed 3 tons per square foot.

Lead Seal.

When lead sealing (*Fig. 18*) is employed the asphalt and steelwork contractors are quite independent of one another, the former being able to carry out his work after the steelwork contractor has erected the lower steelwork. A $\frac{3}{4}$-inch-thick mild-steel plate, coated on top with bituminous paint, is fixed to the slab base or grillage by countersunk bolts, after which a 5-lb. lead sheet, projecting 9 inches on all sides, and a $\frac{3}{8}$-inch-thick mild-

Fig. 18.



MATERIAL IN BASES:
Bloom base and flange angles as necessary
$\frac{5}{8}$" thick base-plate riveted to angles.
$\frac{3}{4}$" thick plate, bottom arrises rounded off.
5 lb. lead sheet to project 9" on all sides of $\frac{3}{4}$" plate.
$\frac{3}{8}$" thick plate with top arrises rounded off.

steel plate, coated on the underside, are set-screwed to a $\frac{3}{4}$-inch plate. The stanchion is then erected with holding-down bolts screwed tightly to the slab or grillage. The projecting lead sheet is then turned up on all sides, and the concrete grillage-casing is cast up to the underside of the $\frac{3}{4}$-inch plate. Later, two-coat asphalt is laid up to edge of the $\frac{3}{4}$-inch plate, and the lead sheet is turned down and carefully beaten on to the asphalt, after which the top surface is coated with bitumen. The last coat of asphalt is then laid over the asphalt and lead sheet, and a $\frac{3}{4}$-inch-thick skirting is carried 12 inches up the stanchion. The concrete floor slab is then cast. With this method it is essential to ensure that all the bolts are tightly screwed in bitumen, and that the lead is compressed, thus preventing leakage at the bolt-holes.

In cases where the asphalt is carried under the grillages, the basement slab can be tied down to the foundation by U-bars under the grillages. This is impossible in the case of the lead seal, as the bars would pierce the

asphalt. The uplift of the slab is taken up by the 4-inch-thick reinforced-concrete casings round the stanchions up to the underside of the floor-slab above.

The great drawback with asphalt is the difficulty of tracing and repairing leaks; water has been known to penetrate the concrete lining as far away as 20 feet from the actual crack in the asphalt. There are various brands of waterproofing cements on the market, which are applied as renders. These are expensive, however, but as they adhere very well to the concrete, it is unnecessary to have a reinforced-concrete lining; consequently, any subsequent leak can be quickly noticed and repaired.

Waterproofing compounds can also be added to well-graded concrete, but particular care must be taken in mixing, and at construction joints, as any faulty workmanship makes their use ineffective.

It is not intended to cover all types of foundations in this Paper, the methods outlined being applicable only where compact strata with good bearing qualities have to be contended with. In cases where the soil is poor, or excessive water is present at shallow depths, it is usually necessary to resort to reinforced-concrete piles or rafts. The former can be divided into three classes: driven pre-cast piles; driven cast-in-situ piles; and bored cast-in-situ piles. Considerable care must be taken if driven piles are used, as the vibration during driving might endanger the adjacent properties. It is, therefore, advisable to obtain the adjoining owner's permission to drive piles, as injunctions might be served to restrict the noise at certain times, if not completely, thus holding up the job and necessitating resort to different methods.

Recently, new processes have been developed, such as chemical consolidation of sandy subsoils, and water-lowering by sunken wells, both of which were used with considerable success in the construction of a large multiple store, thus enabling the new foundations and asphalt tanking to be constructed while the existing buildings above were retained as shopping space. Although these processes are, naturally, more expensive than the normal methods at present in use, they permit such a saving in time that, in this particular instance, approximately 3 months of shopping time was saved, which more than compensated for the extra initial cost.

In conclusion, the Author wishes to express his thanks to Messrs. B. L. Hurst & Peirce, Consulting Engineers, for permission to illustrate the details of the foundations, all of which were devised and carried out under their supervision and which, in many cases, embodied special methods devised by them.

The Paper is accompanied by eighteen sheets of drawings, from some of which the Figures in the text have been prepared.

CHARLES HAWKSLEY PRIZE, 1940.

COMPETITORS for the award in 1940 of the Charles Hawksley Prize were invited to submit designs of one of the following engineering structures, fulfilling the requirements stated :—

1. It is proposed to construct a water tower carrying a covered tank, for an urban water supply, with a capacity of 150,000 gallons, the bottom of the tank being 75 feet above ground level. The ground below the tower is clay overlying rock 25 feet below.

2. It is proposed to construct, at the intersection of two main roads in a built-up area, a traffic roundabout having an underground garage which might, in emergency, be used as an air raid shelter. The roads are 85 feet wide (dual carriageways each 30 feet wide, two footways each 10 feet wide and a centre division 5 feet wide). The central island roundabout is to be, in area, approximately equivalent to that of a circle 240 feet in diameter. The shape of the island may be varied from a circle to improve the traffic facilities. It is to be surrounded by a carriageway 40 feet wide, with an inner footway 3 feet wide and an outer footway 12 feet wide.

The car park and shelter may extend under the carriageway of the roundabout, and should be so designed as to give easy access to pedestrians and motor vehicles.

There is blue clay at a depth of 20 feet below ground-level, with a 3-foot bed of gravel above it and made-ground above the gravel.

Drawings submitted should include a general plan, cross-section, details of columns and girders, and particulars of foundations and retaining walls, with perspective sketches showing entrances, and one view taken from a point which would give the most extensive view of the interior.

Calculations of columns, floors, beams, walls and foundations should also be shown. The outer walls, roofs and floors should be of watertight construction. Special regard should be paid to the capacity of the underground structure, which should be of the multi-floor type, and the roof should be designed from an A.R.P. standpoint.

On the report of the judges, the Council awarded the Prize to William Eugene Blackmore, Stud. Inst. C.E., of Colchester, for his design of a water tower.

Mr. Blackmore has furnished the following note giving details of the basis of his design, together with illustrations of a perspective view and a sectional elevation and plan.

Design for a Water Tower.

By WILLIAM EUGENE BLACKMORE, Stud. Inst. C.E.

The requirements of the problem fixed the capacity and height, but left open the choice of material and æsthetic effect.

Reinforced concrete was selected on account of its clean and solid appearance and its adaptability to any shape. Simple straight lines and flat surfaces were adopted as a basis for the design, simplifying the shuttering and harmonizing better than curves with the type of buildings usually found in the outskirts of an urban area. This led directly to a polygonal

tank, the number of sides being fixed at eight when the shape of the substructure was chosen.

The essentials of the design of the supporting column were: resistance to buckling, lateral stability, and simple shuttering. The section selected, shaped like a broad cross, met these conditions and at the same time provided a pleasant combination of shadows.

Stresses for the strength of the reinforced concrete were based on the D.S.I.R. Code[1], and for the water-retaining qualities of the concrete on the Inst. C.E. "Code of Practice[2]." The tank presented a complex structural problem, the walls being subjected to bending in one direction and to combined bending and tension in another. The calculations led to a fairly simple arrangement of bars, the main reinforcement being placed horizontally in the outer walls to resist hoop tension and radially in the bottom to resist the fixing moment. For the foundations it was found to be more economical to take piers down to rock-level than, for instance, to distribute the load over the clay by means of a raft.

Such a water-tower would be suitable for erection in practically any environment, but it was visualized more particularly in a landscape similar to that illustrated in ***Fig. 1.***

---

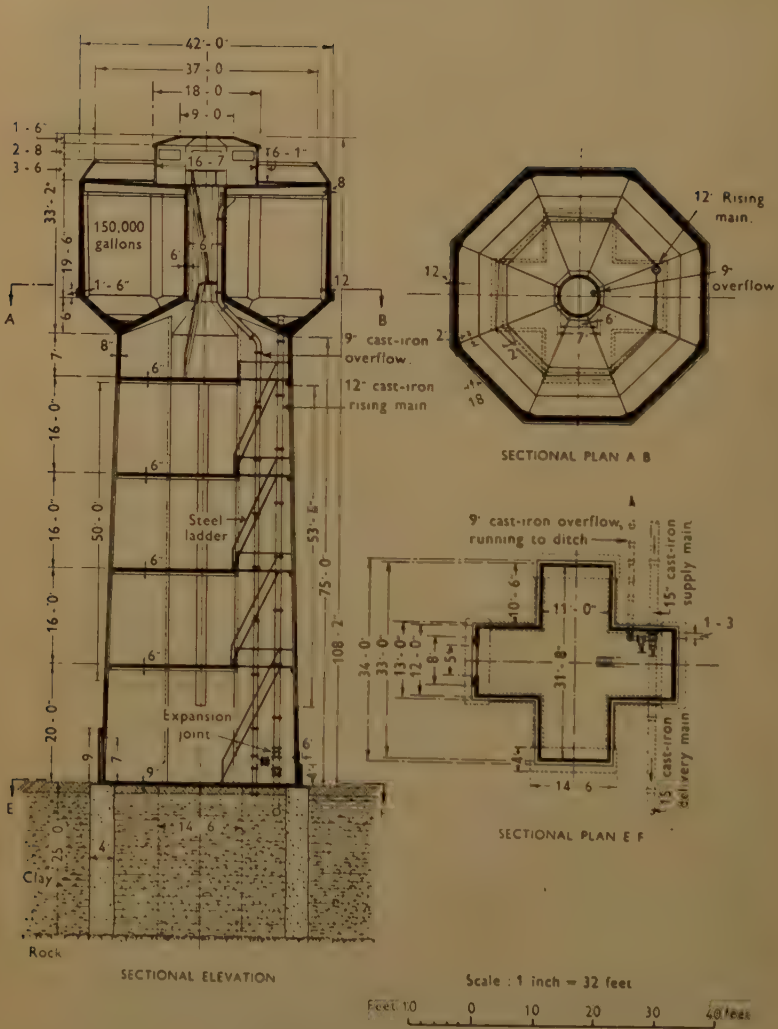
[1] "Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings." H.M. Stationery Office. 1934.

[2] "Code of Practice for the Design and Construction of Reinforced-Concrete Structures for the Storage of Liquids." 1938.

Fig. 1.

DESIGN FOR A WATER TOWER.

Fig. 2.



Design for a Water Tower.

New Works in Progress by the Government of Madras Electricity Department in 1940.

Contributed by Lieut.-Colonel M. G. PLATTS, C.I.E., O.B.E., M.C., B.Sc., M. Inst. C.E.

THE PAPANASAM HYDRO-ELECTRIC DEVELOPMENT.

THIS development, of about 27,000 horse-power, in the Tinnevelly District is under construction. The transmission system is designed to serve the area south of Madura and to tie in with the Pykara system there. The scheme consists of a reservoir of 5,500 million cubic feet capacity on the Thambraparni river, formed by a masonry gravity dam 170 feet in height above the river bed. From this a controlled supply will flow along the course of the river to the diversion weir, 6 miles downstream, which is a masonry overflow structure about 35 feet in height and 1,350 feet in length, designed to pass a flood of 120,000 cusecs. Two 9-foot diameter steel low-pressure pipes 3,500 feet in length conduct the water from the intake here to the surge tank and penstock, from which it drops through a head of 330 feet in four 69-inch diameter pipes to the power-house. Each penstock pipe will serve one vertical Francis turbine of 8,150 horse-power, driving a 7,250-kilovolt-ampere, 11,000-volt alternator. The scheme also includes a steam power-station to be built later. The estimated cost of the full development is £1,700,000.

Only one low-pressure pipe and three penstock pipes and generating units are now being installed, and the cost of this first stage is about £1,300,000.

Work was begun on camps, roads, construction, power-supply, etc., in June 1938, and by August 1940 the following progress had been made :—

The dam foundations had been completed and about one-quarter of the structure built. The two 8-foot 6-inch diameter discharge pipes had been fabricated and were being built in. The diversion-weir foundations had been completed and the full section built for a length of about 800 feet. The intake chamber was completed and the gates and screens were under erection. The 3,500-foot length of 9-foot diameter low-pressure pipe had been fabricated in the contractor's workshop at site and erection was almost completed. Fabrication of the three 69-inch diameter penstock pipes was just being begun, and the pipes were expected to be ready for erection by December 1940. Work on the power-house building has been delayed owing to late delivery of steel-frame sections and of the built-in parts of machinery. These are expected by the end of the year, and it is hoped that the plant and transmission system will be in commercial operation by the end of 1941.

The constructional works are being executed by departmental administration. The following are the principal contractors for machinery and equipment :—hydraulic controls for dam and diversion weir, Ransomes & Rapier, Ltd.; pipes, The Indian Hume Co.; surge-tank and butterfly valves, etc., Boving & Co., Ltd.; water-turbines, The English Electric Co., Ltd.; generating and control equipment, the British Thomson-Houston Co., Ltd.; and the 66-kilovolt transmission-lines, Callender's Cable and Construction Co., Ltd.

The principal load centres will be Madura, Tinnevelly, and Tuticorin.

In addition to the Papanasam scheme the following additions to other plants are now in process of manufacture, and will be installed as soon as deliveries can be effected.

The Mettur Hydro-Electric System.

A fourth 16,000-horse-power unit is being installed; this is similar to the three existing sets installed in 1937, and consists of twin horizontal Francis turbines driving a 12,500-kilovolt-ampere alternator generating at 11,000 volts. The turbines are designed to operate under heads ranging from 60 feet to 160 feet.

They are being supplied by the English Electric Co., Ltd., and the generators and main electrical equipment by the Metropolitan-Vickers Electrical Co., Ltd.

The Andhra Power System.

The steam power-station at Vizagapatam harbour is being extended by the addition of a third 1,500-kilowatt turbo-alternator set supplied by the British Thomson-Houston Co., Ltd., and the steam power-station at Bezwada by a third unit of 3,000 kilowatts manufactured by Messrs. C. A. Parsons & Co., Ltd., together with a Babcock & Wilcox boiler of 33,000 lb. rating.

A major extension to the Pykara hydro-electric power plant was also completed and put into operation in 1939. This consists of two Pelton-wheel units served by a third penstock pipe ranging from 42 inches to 37½ inches diameter. The head of 3,100 feet is believed to be the highest in the British Empire. The water-wheels are of 16,000 horse-power each, running at 600 revolutions per minute and driving 12,500-kilovolt-ampere alternators generating at 11,000 volts. The contractors for the penstock were Messrs. Boving & Co., Ltd., acting for Messrs. Ferrum. The Pelton wheels were made by the Escher Wyss Engineering Works, Ltd., and the alternators and transformers, etc., by the Metropolitan-Vickers Electrical Company, Ltd.

OBITUARY.

SIR ROBERT ABBOTT HADFIELD, Bart., F.R.S., was born at Sheffield on the 28th November, 1858, and died at Kenry House, Kingston Hill, Surrey, on the 30th September, 1940. After his technical training in the Hecla works of Hadfield's Steel Foundry Company, he occupied various responsible positions in the firm, and on its conversion into a limited company became chairman and managing director. His invention of manganese steel, after 10 years of research work, opened a new field in metallurgy, whilst he was also responsible for the discovery and development of silicon steel and many other special alloy steels, and for numerous metallurgical improvements. His work received world-wide recognition and he was awarded almost every medal possible from scientific bodies in Europe, America, and Asia, including the Bessemer gold medal of the Iron and Steel Institute and the John Fritz gold medal of the four great American engineering societies. He was also appointed a Commander of the French Legion of Honour. In 1899–1900 he was Master Cutler of Sheffield. He received the honour of knighthood in 1908, and became a Fellow of the Royal Society in 1909. On the outbreak of the Great War in 1914 he and Lady Hadfield, C.B.E., established and maintained at their own expense the Hadfield Hospital at Wimereux, near Boulogne, which received about 20,000 officers and men. From 1915 to 1918 he was a member of the Advisory Panel of Munitions, Inventions Department. He was created a baronet in 1917.

Sir Robert was a member of many British and foreign scientific societies, and served as President of the Iron and Steel Institute, and of the Faraday Society. He was a prolific writer, and, in addition to several books on metallurgical subjects, was the Author of more than 200 Papers contributed to technical societies.

He was elected an Associate Member of The Institution on the 1st March, 1887, was transferred to the class of Member on the 21st January, 1896, became a member of Council in 1904, served as Vice-President from 1935 to 1937, and was elected an Honorary Member on the 23rd March, 1937. For Papers read before The Institution he was awarded the Howard Quinquennial Prize, the George Stephenson gold medal, the Telford gold medal, and Telford premiums. In 1906 he delivered the "James Forrest" Lecture, his subject being "Unsolved Problems in Metallurgy."[1]

In 1894 he married Frances Bett, daughter of Colonel S. M. Wickersham, of Alleghany, Pennsylvania. With his death the baronetcy became extinct.

[1] Minutes of Proceedings Inst. C.E., vol. clxvi (1905–6, Part 4), p. 190.

CHARLES HESTERMAN MERZ was born at Gateshead on the 5th October, 1874, and died as the result of enemy action on the 15th October, 1940. He was educated at Bootham, York, and at Armstrong College. Throughout his career he was intimately connected with the development of electric power-supply and electric traction, including Newcastle-on-Tyne (1900), involving the first use in England of three-phase distribution at the then high pressure of 6,000 volts, and the design and construction of the Neptune Bank power-station; the electrification of the Tyneside lines of the North Eastern Railway, and the design of the Carville power-station (1903); promotion of the London power bill, and electrification of collieries and iron and steelworks in the north of England (1905); electrification of suburban railways at Melbourne (1907), Buenos Aires (1909), and Bombay (1913), followed by extension of the electrification of the railway over the Ghats; the Carville "B" power-station and the North-Tees power-station (1914–1918); and many other power schemes in America and South Africa. During the Great War he was Director of Experiment and Research at the Admiralty, dealing especially with anti-submarine warfare. In 1919 he advised upon the design of the new large power-station at Barking. He was a member of Government committees dealing with electricity in mines, power-supply, fuel economy, etc., and in 1917 was chairman of the electric power-supply section of the Haldane Coal Conservation Committee. In 1925 he put before the Electricity Commissioners a report which led later to the Act of 1926, creating the Central Electricity Board, and to the construction of the "Grid." In 1932 he was awarded the honorary degree of D.Sc. by Durham University. In 1937 he was Chairman of an international Commission appointed by the Egyptian Government to inquire into the utilization of water-power from the Aswan dam.

Dr. Merz was elected a Member of The Institution on the 10th January, 1905. He was Vice-President of the Institution of Electrical Engineers from 1912 to 1915, and was awarded the Faraday medal in 1931. He was also a Fellow or member of numerous other technical societies, to which he presented Papers on power-supply and railway electrification. His brilliant brain conceived many ideas of a creative nature for the benefit of industry and mankind as a whole, and his services were given unstintingly without thought of self or of remuneration to all who called upon him—and especially to the Government and the national service. He was keenly interested in the younger generation, and was always ready to listen to their point of view, whilst his guidance and advice assisted many of them to make the right choice of a career.

In 1913 he married Stella Alice Pauline Byrne, daughter of Edmond de Satur, of Dublin, and had one son and one daughter, both of whom lost their lives with their father. His son, Robert, was engaged on work of national importance, in spite of his youth, and had already shown full promise of outstanding ability to follow in his father's footsteps.

ABSTRACTS OF THE CURRENT TECHNICAL LITERATURE OF ENGINEERING AND APPLIED SCIENCE.

ENGINEERING CONSTRUCTION.

The Application of Soil Mechanics in Building the New York World's Fair. G. L. FREEMAN, G. W. GLICK, and H. GRAY, jun. (**Civ. Engng., N.Y.*, 10, *649–652; Oct. 1940*).—The reclamation of the site for the fair afforded an unusual opportunity for soil studies and the application of soil mechanics. The Authors review a number of the problems involved in the work, describe the field tests, laboratory studies, and mathematical analyses of soil behaviour, and discuss the solutions developed. They also describe the fill-placing methods, and the foundation designs adopted to meet the difficult conditions encountered. They state that, generally, where the principles of construction determined by soil mechanics were followed, the results were successful and satisfactory; but so many factors have intervened that there has been little opportunity to check forecasts against actual performance.

The Pressure of Earth against Lateral Supports. R. R. MINIKIN (**Engineer, Lond.*, 170, *244–246; 18 Oct. 1940*).—The Author describes a simple testing apparatus devised by him, and the manner in which pressure readings are obtained. It consists essentially of a rectangular box 5 feet by 2 feet by 1 foot 6 inches wide, with plate glass sides and timber bulkheads capable of extension with wind sides for surcharge. To measure the pressure of the sand, a form of gauge is employed, consisting of two plates of plywood with chamfered edges having three small sponge rubber disks between them. Various tests made with the apparatus are described, and simple graphical constructions are given for finding the triangle of the wedge of rupture. A comparison of the values given by the Author's formula with those of Rankine's formula and of the Jenkins revised wedge theory is tabulated.

Foundation Exploration at Kentucky Dam. A. V. LYNN and R. F. RHOADES (**Engng. News-Rec.*, 125, *480–483; 10 Oct. 1940*).—For more than 4 years prior to the start of constructional work, in July 1940,

---

NOTES.—An asterisk prefixed to a reference, thus **Civ. Engng., N.Y.*, denotes that the article is illustrated.

The abbreviated titles of periodicals are those used in the "World List of Scientific Periodicals" (Oxford, 1934).

on the dam, which is situated on the Tennessee river 22 miles above its confluence with the Ohio river, an extensive exploration was made of the site, involving core drilling, wash boring, the sinking of test-pits, the sampling of soils, and a thorough co-ordination of engineering and geological observations. The dam, a concrete and earth-fill structure, will be 8,650 feet in length and will create a reservoir 73 feet above the present river-level. The drilling procedure is described in detail, and typical records of boring explorations are reproduced.

Core Control and Cutoff Construction at Kingsley Dam, Nebraska. W. J. Turnbull and G. N. Carter (**Civ. Engng., N.Y.*, **10**, *623–627 ; Oct. 1940*).—The Kingsley dam is a hydraulic fill structure about 8 miles north-east of Ogalala, Neb., constructed for the purpose of storing flood-water for power and irrigation. The main structure is about 2 miles long, and the dike adjoining it is about 1 mile in length. In the main section the base-width is 1,140 feet and the height 160 feet. The main borrow-pits contained only about 30 per cent. of the fine material needed for core construction. Mr. Turnbull describes the means employed to make up the deficiency from the sandy and silty loess forming the surface deposit on a ridge extending to the southward from the south abutment. Mr. Carter describes the construction of a concrete curtain-wall in one abutment. This involved driving a 600-foot tunnel into the hillside and sloping it down time and again as successive lifts of the wall were formed, until a curtain 100 feet high had been completed.

A Brief Review of Steel Column Tests. L. T. Wyly (**J. Western Soc. Engrs.*, 45, *99–113 ; June 1940*). The Author reviews and analyses the data furnished by tests on column models from those made for the second Quebec bridge in 1908 to date. He presents the following three propositions, and discusses their implications : (1) the majority of structural columns are short columns having an L/r ratio under 50, for which the Euler loads have little significance ; (2) about three-quarters of (American) laced structural columns fail through local weakness at a unit stress below the yield-point of the material, and thus do not conform to the assumptions of the column theory ; (3) improvements in column design to provide against local failure result in increased column efficiency. A classification of failures is tabulated, and examples of failure are illustrated. Finally the Author suggests various means of avoiding local weaknesses.

Plain and Reinforced Concrete Arches. (**J. Amer. Concrete Inst.*, **12**, *1–26 ; Sept. 1940.*)—A report of a Committee of the Institute summarizes research work carried out since 1932 and presents a suggested specification for the design of reinforced-concrete arches. Investigations of the effects of shrinkage and plastic flow are described. The results of

measurements made on long arches at Pittsburg, Pa., are discussed. Recommendations are made as to the calculation of moments and thrusts due to loading and volume change. Finally, a new method is proposed for the design of arch ribs, based upon ultimate strength formulas, and the problem of the factor of safety for members subjected to direct load and flexure is considered.

Effect of Warping upon Continuous-Truss Bridges. L. T. Wyly, T. I. Fullenwider, and R. B. Murphy (**Engng. News-Rec.*, 125, *280–282 ; 29 Aug. 1940*).—In adjusting the reactions of two continuous bridges over the Illinois river, at Hennepin and Lacon, Ill., on the basis of proving-ring measurements, it was found that warping of the spans exerted a considerable effect upon the stresses. The bridges are similar, each carrying two lanes of traffic on concrete decks. Each has a continuous-truss unit of three spans (324 feet, 378 feet, and 324 feet), whilst the Hennepin bridge has also a continuous truss of two 300-foot spans. Two proving-rings, each of 300,000-lb. capacity, were used in the measurement, and a high degree of precision was achieved by careful control of temperature, moisture, and wind-pressure on the rings. The results are plotted in curves.

Underpinning a Tall Building in Chicago. (**Engng. News-Rec.*, 125, *422–426 ; 26 Sept. 1940.*)—The seventeen-story Monadnock building is 66 feet by 400 feet in plan, the long dimension facing Dearborn and Federal streets. As it was feared that construction of the city's new subway under Dearborn street might endanger the building by causing settlement of the ground under the floating foundations upon which it was erected, the front row of piers along Dearborn street is being picked up by jacks on grillages while Chicago-type well caissons are sunk 60 feet to hardpan directly beneath the piers, after which the building load is transferred to the caissons. The operations are described in detail. The caissons are 4–5½ feet in diameter and are belled out at the bottom from 9 feet 3 inches to 13½ feet diameter. In six cases where two piers rest on a common mat, a heavy reinforced-concrete girder is built between the tops of the caissons and the piers, the two caissons jointly carrying the two piers.

A Method of Re-aligning and Transitioning Railway Curves. S. R. Chopra (**Quart. Tech. Bull. Indian Rly. Bd.*, 5 (No. 58), *9–12 ; July 1940*).—The curve to be re-aligned is divided into chords of length equal to one-quarter of the length of the transition curve ; the ends of these chord-lengths are designated " stations." The method consists of two stages. In the first an imaginary circular curve is assumed to be located having the same tangents as the tangents of the distorted curve, and its vertex is moved from the vertex of the distorted curve away from the

centre by an amount equal to the shift determinable from the radius of a circular curve passing through the vertex of, and tangential to, the tangents of the distorted curve. In the second stage the final curve is located at the various " stations " on the circular curve so that the final curve overlaps approximately the middle portion of the distorted curve. The Author states that the method can be utilized also in cases where bridges, level crossings, heavy cuttings, or high banks limit the sluing of the track at certain points of the curve.

Subsoil Water Problems at a Railway Underpass. (**Rly. Age, Chicago*, **109**, *366–368 ; 373 ; 14 Sept. 1940.*)—When the Norfolk and Western Railway constructed an underpass recently at Norfolk, Virginia, involving depression of the street-level by a maximum of about 11 feet below the subsoil-water table, unusual measures were required for unwatering the subsoil to enable construction to proceed, and also for dealing with the water after completion of the works. The railway bridge over the underpass carries six tracks. It is placed on a skew of 25 degrees and has a clear span of 41 feet. The footings for the wing walls and the abutments are supported on timber piles, whilst three reinforced-concrete struts extend between the abutment footings, under the street pavement, one at each end of the subway and the third near the centre. The water problem was solved by the adoption of an extensive well-point system, and the permanent measures included subsoil drains leading to a sump from which the water is removed by automatically-controlled motor-driven pumps. The various operations are described in detail.

Water Power in Eire. (**Electrical Review*, **128**, *237 ; 20 Sept. 1940.*) —A brief description is given of the Liffey hydro-electric project, now under construction, comprising a dam about 100 feet high above the falls at Poulaphouca, serving two power-stations, at Poulaphouca and at Golden Falls. Some details of reservoir capacity and discharge, and of electrical output, are given, and the progress of constructional work is summarized.

MECHANICAL ENGINEERING.

Modern Measuring Instruments and the Principles of their Design. G. Schlesinger (**J. Instn. Production Engrs.*, **19**, *317–358 ; Sept. 1940*). —The Author discusses the refinements of fit and limits, the possibilities of accurate measurement, the character and range of instruments, the direct reading and estimation of fractions of divisions, and the formulas for calculating the degree of magnification. The measuring system may be based upon mechanical or optical means, micro-comparator gauges, or methods of " active " calibration. The various instruments are described in detail. The modern tendency is to avoid the separation of machining

and measuring and to combine the controls for the cutting-tool or grinding-wheel, with the measuring apparatus. This is achieved by the use of a diamond feeler, or a photo-cell, or by controlling the cutting-tool directly from a gauge or fixed standard incorporated in the machine.

A New Type of Power-Torque Meter. (**Soc. Nav. Arch. Mar. Engrs., Adv. pf., 7 pp.; Nov. 1940.*)—The Author describes an experimental meter developed at the Naval Research Laboratory, Washington, D.C., which includes means to indicate instantaneous values of shaft torque, shaft power, and shaft speed. The design is an adaptation of the usual type of short base-length torsion-meter, and utilizes the property of the electrical micrometer tube, that the meter reading for a given displacement of the tube-arm, within certain limits, is exactly proportional to the voltage applied to the bridge. A schematic diagram of the electrical micrometer tube in its bridge network is reproduced, and the construction of the instrument is described in detail. Comparative measurements obtained with the meter are plotted in curves.

A New 15-Kilovolt Pneumatic Circuit-Interrupter. L. R. Ludwig, H. L. Rawlins, and B. P. Baker (**Elect. Engng., N.Y.*, 59, *528–533; Sept. 1940*).—After a brief historical review of the development of oil-less circuit-breakers, the Authors emphasize the applicability of compressed air, and describe a new type which can attain an interrupting capacity of 1,500,000 kilovolt-amperes at 15,000 volts. Its principal features are an extremely high operating speed, and rapid interruption with minimum energy-dissipation. Test results are presented in the form of oscillograms. These show that currents of 65,300 amperes have been interrupted at 13,200 volts, single-phase, with a contact separation of 2 inches, which can be obtained mechanically in one half-cycle. A bibliography of the subject is appended to the Paper.

Magnetic "De-Ion" Circuit-Breaker for 2,500–5,000 Volts. L. R. Ludwig and R. H. Nau (**Elect. Engng., N.Y.*, 59 (Trans.), *518–522; Sept. 1940*).—The Authors discuss the theory of the quenching of an arc by de-ionizing the normally conducting arc-stream by means of a gas-blast produced by an intense transverse magnetic field, and describe a new form of air circuit-breaker consisting of a large number of non-gas-forming insulating plates having V-shaped slots which are spaced apart and placed at right angles to the arc-path. The problem of adequately enclosing the breakers is discussed, and test results are presented, which indicate that excellent interruption can be obtained in an enclosure of small overall dimensions.

The Making of Comparative Efficiency Tests with Locomotives on the Road. C. A. Cardew (** J. Instn. Loco. Engrs.*, 30, *294–356; July–*

Aug., 1940). The Author deals with the subject under the following headings: (1) general considerations relating to test conditions; (2) coal and water supplies, and their measurement; (3) testing apparatus, and factors affecting accurate results in its design and application; (4) details which affect the collection of truly comparative data; (5) the tabulation of mean test results and the making of deductions therefrom relative to the comparative efficiency of performance; and the determining influence of economic factors. He states that tests made without a dynamometer-car can yield surprisingly accurate data and reliable results. The various aspects of the problem are discussed in detail, and the test results are presented in Tables and curves.

Power-Transmission by Belting. J. G. Jagger and F. Sykes (**Proc. Instn. Mech. Engrs.*, **143**, *318–327; Oct. 1940*).—In the course of experimental work on flexible belts running on pulleys of small diameter, it was observed that the loss of speed of the follower arising from the elasticity of the belt was considerably greater than would normally be expected. It was also noticed that this loss of speed depended upon the mean tension of the belt and the diameter of the pulley. The Authors discuss possible causes for the deviation of the creep from that given by the simple elastic law. They tabulate experimental values of creep for a fabric belt, and plot in curves the results obtained for elastic and for fabric belts. They describe experiments made, on belts of a special endless woven construction, which ensured that no errors would be introduced owing to lack of uniformity at the fastening, and give the results obtained on a belt 4 inches by ¼ inch. They discuss surface compression, stress-distribution, the efficiency of flat belts, the effect of unequal pulleys, and the efficiency of V-belts. Finally they present general conclusions based upon the theory which assumes no compressive stress, observing that the extent to which these will apply in practice to any particular belt material will depend upon how closely the assumed conditions hold.

The Butt-Welding of Steel Tubes and Pipes. H. Harris, J. E. Jones, and A. L. Skinner (**Trans. Inst. Welding*, 3, *115–156; July 1940*).—The Authors describe an extensive investigation on the general problem. They state that the main defects due to the butt-welding of steel tubes and pipes are "basal cracks"—small or even minute cracks in the region of the root of the weld. The tests made included electric welding with copper and with steel backing rings, and with no backing rings; the results are illustrated by numerous micro-photographs. The Authors discuss factors influencing the formation of basal cracks, including the electrode; the design of joint preparation; the temperature of the pipes being welded; the proficiency of the welder and the welding procedure; and the metallurgical character of the pipe steel. They also discuss the applicability of gas welding, and describe the welding technique.

Finally they deal with preheating, stress relief, and normalizing, and summarize experience in regard to methods for the examination of pipe butt-welds, made with the object of establishing a welding technique or of testing the proficiency of a welder.

MINING ENGINEERING.

The Pressure exerted by the Roof upon the Coal-Seam near the Face in Workings advancing to the Strike. F. K. T. van Iterson (*Proc. Koninkl. Nederlandsche Akad. van Wetenschappen*[1], 43, *149–158; 294–306; 412–424*).—The following problems are analysed mathematically and discussed by the Author: (1) the stress in the roof and in the floor near the face, whilst considering the coal as uniformly resistant matter compressed between perfectly lubricated roof and floor; (2) the distribution of the pressure around hollow spaces in the homogeneous massif according to the laws of elasticity; (3) the pressure of the roof on coal pillars; (4) the pressure of the roof on the coal-seam, the coal being considered as perfectly plastic matter; (5) the pressure of the roof and the floor on the coal-seam, the coal seam being considered as pulverulent matter.

Roof Movements and their Control on Some Conveyor Faces. J. R. Dinsdale and J. M. Hughes (**Proc. S. Wales Inst. Engrs.*, 56, *169–187; 24 Sept. 1940*).—The research described was carried out at twelve collieries and in fifteen districts in order that a representative range of conditions could be studied. Most of the observations were made on double unit conveyor faces of such length that the effect of the ribs at each side of the district was almost negligible, and in seams where the floor lift was so small in comparison with the roof subsidence that the reading on the convergence-recorder used could be taken for all practical purposes as roof subsidence. The thickness of the seams ranged from 2 feet to about 7 feet, and their depth from about 100 yards to 900 yards. The procedure is described and the results are plotted in curves. The measurements showed that pressure-cycles of varying intensity occurred on almost every face. These conditions resulted in considerable damage to timber and steel supports and made withdrawal of face posts a difficult operation. It is concluded that, to avoid this danger and to effect the maximum economy in supports, the resistance of both temporary and permanent supports should be sufficiently high, even in thin seams, to keep convergence to the minimum and at a uniform rate in order to eliminate cycles of pressure.

Experiments with Reinforced-Concrete Props. J. K. Coultas and H. Henshaw (**Colliery Engng.*, 17, *225–226; Sept. 1940*).—The Authors

[1] The Paper is in French.

describe tests made with the object of determining the ability of reinforced-concrete props to withstand repeated setting and withdrawal. The seam was 3 feet 6 inches thick, at a depth of 500 yards, and the props were set on a double-unit strike face 285 yards long, with a dip of 1 in 7. Packing and withdrawal of supports was done on the afternoon shift, followed by coal-cutting and shotfiring on the late afternoon and night shifts in preparation for the day filling shift. The roof was effectively controlled under these conditions and the height reduction during the time a prop was set was about 6 inches—conditions favourable to the experiment. Eighty props were used. The release device, which had no mechanical parts, was a steel tube 8 inches in diameter and 8 inches high, weighing 10 lb. Its operation is described. The experiments demonstrated that reinforced-concrete props set with release devices can be used over and over again on working faces with strong floors under normal roof conditions.

Coal Washing at a Midland Colliery. (**Colliery Guardian*, 161, *347–351 ; 11 October 1940.*)—A Barvoys plant has been installed for the cleaning of the Deep Soft seam mined in the Midlands. The seam, whilst mainly of low ash quality, has a small hard coal band which it is desirable to remove, as the coal is largely sold for domestic fuel. A detailed description of the working of the plant is given. The cleansing medium consists of a suspension of barytes and clay in water. The raw coal is fed to the surface of the bath. The clean coal floats on top and is removed by a scraper ; the shale sinks to the bottom and is removed by an elevator ; the middlings product is extracted by means of a tube in which flows an upward current of medium, to a tank in which the medium is passed off through a screen. The clean coal, the middlings and the shale are sprayed with water to recover the barytes. The clean sprayed coal is screened into two sizes, 3½ inches to 1⅞ inch, and 1⅞ inch to ⅞ inch. The sprayed middlings are crushed and reduced to less than 1⅞ inch, and are then screened into two sizes, above and below ⅞ inch. The plant was designed to deal with 100 tons per hour, with a peak-load capacity of 120 tons per hour.

The Estimation of Firedamp. C. S. W. Grice and D. W. Woodhead (**Colliery Guard.*, 161, *207–208 ; 30 Aug. 1940*).—The Authors describe two forms of apparatus, developed by the Safety in Mines Research Board for instructional purposes in demonstrating gas caps in known percentages of gas. These are free from electrical or mechanical parts requiring frequent adjustment. The products of combustion are kept separate from the firedamp-air mixture. The flame is easily adjustable while the lamp is in the apparatus. Lamps of different types can be compared in the same mixture. In one apparatus, which has a row of chambers, the caps produced in various mixtures of firedamp and air can be shown simultaneously to several observers.

Stemming in Metal Mines. J. A. JOHNSON, W. G. AGNEW, and M. MOSIER (*U.S. Bur. Mines, R.I. No. 3509, 27 pp., June 27 1940*).—The object of an investigation in progress at the Mont Weather testing adit on stemming used in connexion with blasting is to determine, under controlled conditions, for each of several classes of stemming and for no stemming, the quantities of the noxious gases formed, the quantity of dust produced, and the efficiency of the explosive. Investigations are being made with (1) no stemming; (2) sand in two paper cartridges; (3) clay in two paper cartridges; (4) commercial hydrated lime in two paper cartridges; (5) combustible blasting-plugs; (6) ceramic blasting-plugs with clay in two paper cartridges. The Authors describe the test procedure and present an analysis of the results on dust studies from fifty-nine rounds blasted in the adit. It was found that the least dust was produced when no stemming was used. The dust-concentration in general decreases with the fineness of the rock broken and with the increase in the percentage of break of the round.

Simultaneous Shot-Firing at Rippings. H. STAFFORD (*Iron and Coal Trades Rev.*, **161**, *353–354; 11 October 1940*).—A recent amendment to the Explosives in Coal Mines Order, 1934, permits simultaneous shot-firing to be practised at rippings. The advantages and disadvantages of simultaneous shot-firing are enumerated. The Author, in discussing the required equipment, states that low-tension detonators, having uniform characteristics, are desirable; he emphasizes the necessity for check-testing detonators, and the importance of connecting detonators in series in order to avoid differences in the current flowing to each detonator. He advocates the use of two single-core cables, in order to avoid any possibility of short-circuiting, with consequent danger of some shots misfiring and remaining undetected. A dry-battery-type exploder should be used, to give the required current immediately the switch is closed, and to avoid the ignition of firedamp by sparking. A circuit-testing device should be incorporated. Weekly tests on the exploders are advisable. Experience has shown that to attain good results with simultaneous shot-firing, a technique, different from that employed when shots are fired singly, is required, and the Author discusses various practical precautions to ensure successful working.

Dust Suppression. A. NELSON (*Mine & Quarry Engng.*, **5**, *203–204; July 1940*).—The Author discusses the results of Zeiss konimeter investigations made while observing the performance of devices for the suppression of boring dust under practical conditions. Tests were conducted with various forms of dust protection. Dust-traps were found to give uniformly good results. Wet drilling, with the axial water-feed drill, revealed the detrimental effect of air leaking with the water through worn parts of the mechanism into the hollow drill steel. The results when

using a machine incorporating the external water-flush head were encouraging, but the average drilling-rate was below that normally accomplished. Numerous tests conducted with the wet drilling method using foam gave very satisfactory results as regards arresting the dust. The drilling-speeds, however, were generally lower than those obtained with dry percussive drilling. Moreover, it was very difficult to maintain a constant flow of foam to the cutting-edge, and whilst the quantity was sufficient to bind the dust, it was insufficient to clear the hole satisfactorily. The Author suggests the provision of passages of larger area. Tests were also made with an air-operated rotary drill in a sandstone working. With an air-pressure of 76 lb. per square inch, an average boring-rate of 4 inches per minute was obtained, and was accepted as fairly satisfactory. The dust-concentrations showed an improvement over those observed when dry drilling without a dust-trap.

Electrical Plant at Britain's Most Up-to-Date Colliery. (**Elect. Times*, 98, *177–178 ; 12 Sept. 1940.*)—Opportunity was taken to apply the most modern methods and equipment in the recent development of a virgin coal-area of approximately 6,000 acres, yielding good steam and household coals. Electrical plant is extensively employed. The colliery is all-electric in its power services, and power is transmitted from the Company's central power-station, about 12 miles distant, at 22,000 volts, being transformed down first to 3,000 volts, and then to 500, 100, and 15 volts. Details are given of the winder motors and interlocks, of the pit-bottom and haulage layout, of the ventilating arrangements, and of the coal-preparation plant.

CORRIGENDUM

Journal Inst. C.E., vol. 14 (1939–40), p. 234 (April 1940).

Plate 1, facing p. 234.

For scale of miles 100 – 50 – 0 – 100 – 200
read 50 – 25 – 0 – 50 – 100

NOTE.—The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the Papers and Abstracts published.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.

NOTE.—*Pages* [1] *to* [18] *can be omitted when the Journal is bound in volume form.*

NOTICES

No. 2, 1940—41

DECEMBER, 1940

THE INSTITUTION OF CIVIL ENGINEERS

MEETINGS, SESSION 1940–41.

ORDINARY MEETINGS.

Arrangements have been made for the following subjects to be brought forward, on the dates shown below :—

1940.
(1.30 p.m.)
Dec. 17. * "**The Mohammad Aly Barrages, Egypt,**" by A. G. Vaughan-Lee, M. Inst. C.E.

There will be a ballot for the election of new members.

* An abstract of this Paper appeared at p. [8] of the November Journal.

1941.
(1.30 p.m.)
Jan. 21. ** "**The Design of Sewage-Purification Works,**" by H. C. Whitehead, M. Inst. C.E.

There will be a ballot for the election of new members.

** An abstract of this Paper appears at p. [13], *post*.

(*Light refreshments will be served at* 12.45 *p.m. prior to the above meetings.*)

SPECIAL ANNOUNCEMENTS.

MILITARY SERVICE.

VOCATIONAL INSTRUCTION FOR THOSE TEMPORARILY SERVING IN HIS MAJESTY'S FORCES.

The Board of Education having expressed the desire of the Army Council for the establishment of courses for persons under training for various professions in civil life who wish to continue their studies while temporarily serving in His Majesty's Forces, the Institution of Civil Engineers, together with the Institutions of Mechanical and Electrical Engineers, is co-operating with the Board of Education and the Advisory Council for Education in the Forces in the organization of courses in engineering subjects.

During the last war there was a break in the studies of a whole generation of engineers, with serious consequences to the profession; many on attempting to rejoin it had difficulty in resuming their studies and in passing examinations, and were consequently handicapped in completing their professional qualifications. The Council will regard it as an immense benefit if this experience could be avoided in the present war and consider that the provision of courses of engineering study will, should the conditions of the war allow a measure of study, help those whose engineering training has been interrupted to keep in touch with technical knowledge, and in some cases to complete a portion of their engineering qualifications whilst so serving.

It is hoped that it will be found possible to allow students to attend evening classes at a technical college where one is within convenient reach, but where the distance is too great it may be found possible, when a sufficient number of students come forward in large camps, to arrange for the delivery of regular courses of lectures by qualified teachers. There must, however, remain a large number of cases, such as those of outlying and small stations, in which courses of private study, either by guided reading or a modified form of correspondence course, would be the only possible method.

Curricula suitable for guided reading or correspondence courses are now being prepared in the following subjects, of a standard adapted to the examination requirements of the Institutions :—

1. English	I. Mech. E.	Sec. A, Part II (*a*).
	I.E.E.	Part I (1).
2. Fundamentals of Industrial Administration.	I. Mech. E.	Sec. A, Part II (*a*).
	I.E.E.	Part II, Group II (5).
3. Applied Mathematics (or Mechanics)	Inst. C.E.	Sec. A, 1.
	I. Mech. E.	Sec. A, Part I (*a*)
	I.E.E.	Part I (2).
4. Physics and Chemistry (General Science)	I. Mech. E.	Sec. A, Part I (*b*).
	I.E.E.	Part I (3).
5. Electrotechnics	Inst. C.E.	Sec. A, 3.
	I. Mech. E.	Sec. B (c.4).
	I.E.E.	Part I (4).
6. Strength of Materials	Inst. C.E.	Sec. A, 2.
	I. Mech. E.	Sec. B (*b*).
7. Theory of Structures	Inst. C.E.	Sec. A, 4 (*a*).
8. Theory of Machines	Inst. C.E.	Sec. A, 4 (*b*).
	I. Mech. E.	Sec. B (*a*).
9. Steam Engines	Inst. C.E.	Sec. B, Group iii (3).
	I. Mech. E.	Sec. B (*c*.1).
10. Internal Combustion Engines	Inst. C.E.	Sec. B, Group iii (3).
	I.Mech.E.	Sec. B, Group (*c*.2).
11. Hydraulics	Inst. C.E.	Sec. B, Group ii (1).
	I. Mech. E.	Sec. B (*c*.3).
12. Metallurgy	Inst. C.E.	Sec. B, Group ii (3).
	I. Mech. E.	Sec. B (*c*.5).
13. Surveying	Inst. C.E.	Sec. B, Group i (1).
14. Engineering Geology and Mineralogy	Inst. C.E.	Sec. B, Group iii (1).
15. Electricity Supply	I.E.E.	Part II, Groups I and II.
16. Electrical Communications	I.E.E.	Part II, Groups I and II.

There are other subjects included in the examinations of the Inst. C.E., Inst. Mech. E., and Inst. E.E. which are less frequently selected by candidates for which courses could probably be arranged, should there be sufficient demand.

Any Student of the Institution of Civil Engineers or approved candidate for election to Corporate Membership who wishes to prepare himself for Sections A and B of the Associate Membership Examination of The Institution under this scheme should apply to his Commanding

Officer or to the Unit educational officer for information as to the method of procedure and for copies of the curricula.

ARMY OFFICERS' EMERGENCY RESERVE.

The War Office has informed The Institution that applications for registration in the above-mentioned Reserve are now particularly invited from Corporate Members who are over 31 and not more than 45 years of age, with a view to accepted applicants being subsequently granted Emergency Commissions in the Corps of Royal Engineers. This requirement is therefore brought to the notice of those members who are free and feel disposed to offer their services.

Particulars of the Abridged Conditions of Service, etc., in regard to the Army Officers' Emergency Reserve may be obtained on application to the Secretary of The Institution.

Corporate Members over 45 and under 55 years of age may also apply for registration, but the prospect of employment cannot yet be foreseen.

SPECIAL ENLISTMENT IN THE ROYAL ENGINEERS.

The War Office has notified The Institution that Associate Members and Students of The Institution who are over 23 and under 31 years of age are still eligible for special enlistment in Training Units for the Corps of Royal Engineers, although they are at present included in the Schedule of Reserved Occupations. Further details and forms of application may be obtained from the Secretary of The Institution.

NATIONAL SERVICE (ARMED FORCES) ACT, 1939.

Students of The Institution who are 20 years of age and who are liable for Service under the National Service (Armed Forces) Act, 1939, must register at a Local Employment Exchange when their age-group is called, and may obtain from the Secretary a form of certificate indicating their connexion with The Institution, which, upon production to the Registration Officer, will, it is anticipated, assist them in being posted to the ranks of the Corps of Royal Engineers or to a technical unit in which their qualifications can be employed.

AIR MINISTRY.

ROYAL AIR FORCE VOLUNTEER RESERVE.

The attention of members of The Institution is directed to the fact that the Royal Air Force requires a number of technical officers for employment on engineering and armament duties. Commissions in the Royal Air Force Volunteer Reserve will be granted for the duration of hostilities to suitable applicants between the ages of 21 and 50 years possessing the requisite personal and technical qualifications. The following are the minimum qualifications :—

Engineer.

(i) Holders of Mechanical Engineering Degrees, or Civil Engineering

Degrees if combined with theoretical knowledge of heat-engines.

(ii) Holders of Mechanical Engineering Certificates, or members of Engineering Institutes who also have 2 years' practical experience.

(iii) Practical mechanical engineers who have served an apprenticeship followed by a number of years' experience in erecting or overhauling internal-combustion engines or aeroplane structures, and with knowledge of the properties of engineering materials.

[Students and Corporate Members of The Institution who are desirous of offering themselves as applicants for Commissions as Engineer Officers are requested to note that, according to information received from the Air Ministry, the standard of theoretical heat-engine knowledge required of degree men approaches degree standard and that practical men are required to have expert practical knowledge of internal-combustion engines: as regards knowledge of the properties and testing of engineering materials, this is not expected to be of metallurgical standard.]

The appropriate form of application (No. 1020), and notes on conditions of service, may be obtained from the Secretary of The Institution or from the Air Ministry, S.7.e/5, Adastral House, Kingsway, W.C.2.

The Secretary will be pleased to furnish certificates of membership of The Institution for attachment to applications.

MINISTRY OF LABOUR.

SCHEDULE OF RESERVED OCCUPATIONS.

The following entry appears in the Ministry of Labour's Schedule of Reserved Occupations:—

Student engineering apprentice or learner—reserved at and above the age of 18 years.

This entry relates only to a man employed in industry or under articles to a professional engineer who produces a certificate from a university or technical institution or from a professional Institution of Engineers to show that he is within two years of the satisfactory completion of a course of study with a view to offering himself for the first time for:—

(i) an Engineering Degree;

(ii) an Engineering Higher National Certificate;

(iii) The Associate Membership Examination of the Institutions of Civil, Mechanical or Electrical Engineers, or the Associate Fellowship of the Royal Aeronautical Society;

(iv) An engineering examination of similar standing to those in (i), (ii) and (iii) above, e.g. the Associate Membership Examination of the Institutions of Marine, Mining and Structural

Engineers, Testamur examination of the Institution of Municipal & County Engineers, Higher Grade Certificate in Gas Engineering.

In so far as The Institution of Civil Engineers is concerned, category (iii) applies to Students who are studying with a view to passing Sections A and B of the Associate Membership Examination within a period of 2 years, and who obtain from the Secretary of The Institution certificates to this effect for production to the Registration Officer of the Local Employment Exchange when they register in their age-group under the National Service (Armed Forces) Act, 1939.

Before a Student is furnished with a certificate, he must send to the Secretary full details of his present occupation, state the dates when he proposes to sit for Sections A and B of the Associate Membership Examination, and satisfy the Secretary in regard to the steps he is taking to prepare himself to sit for such examination.

A Corporate Member who is normally engaged in civil engineering, must register at a Local Employment Exchange when his age-group is called, and should designate himself as a "Civil Engineer" to accord with the Ministry of Labour's "Schedule of Reserved Occupations."

He should obtain beforehand from the Secretary of The Institution a certificate of membership, for production to the Registration Officer.

GENERAL ANNOUNCEMENTS.

SUBSCRIPTIONS.

Members and Students are reminded that subscriptions for 1941 are due on the 1st January, 1941. The present subscription rates are as shown below :—

	Class A. (London Area.)			Class B. (Elsewhere in British Isles.)			Class C. (Abroad.)		
	£	s.	d.	£	s.	d.	£	s.	d.
Members	6	6	0	4	4	0	3	13	6
" (retired)	3	13	6	2	12	6	2	12	6
Associate Members	3	13	6	2	12	6	2	12	6
" " (retired)	2	12	6	2	2	0	2	2	0
Associates	5	0	0	5	0	0	5	0	0
Students	2	0	0	1	10	0	1	10	0

Owing to the increased cost of postage and need for economy in the use of paper, members are urged to make prompt payment of their subscriptions and so save the necessity of a further application.

THE JOURNAL.

The next Number of the Journal will be published on the 15th January.

The Council are prepared to receive short Communications of, say, 2,000 words, accompanied by two or three illustrations, for inclusion in the Journal. Such Communications should be topical in character and might deal, for example, with demolition and reconstruction problems, or with minor constructional details, of a novel character, which would be of general interest to engineers.

MESSAGE OF GOODWILL FROM AUSTRALIA.

An extract from a letter received by the Secretary from the Secretary of The Institution of Engineers, Australia, is given below :—

" . . . May I now express the wish that comes to many of us that your Institution, the mother body of the professional engineering institutions throughout the Empire, will not suffer any material damage in the struggle that is now taking place and will be saved from the onslaught which threatens the 'Old Country' day and night. I am sure the thoughts of all true British engineers are constantly with you at these times."

EXAMINATIONS.

The next Associate Membership Examination is to be held in London and the Provinces during the week commencing the 21st April. Completed applications to attend that Examination should be placed in the Secretary's hands by the 28th February. Students of The Institution are recommended to lodge their applications to attend about a fortnight before that date.

The first Common Preliminary Examination (conducted by the Engineering Joint Examination Board), which now replaced the Institution Preliminary Examination, is to be held in London and the Provinces on the 1st to the 4th April, inclusive. Applications from candidates for admission to Studentship and from candidates for election to Corporate Membership, who have been required by the Council to pass that Examination as a condition of their election, should be placed in the Secretary's hands by the 28th February. Copies of the Rules and Syllabus may be obtained upon application to the Secretary.

C. C. LINDSAY CIVIL ENGINEERING SCHOLARSHIPS.

Regulations for the award of these Scholarships, sanctioned by the Board of Education, may be obtained on application to the Honorary Secretary of the Glasgow and District Association (Mr. William MacGregor, B.Sc., Engineering Department, The University, Glasgow, W.2). Eligibility for the award of these scholarships, which are each of the value of not less than £25 per annum, is confined to Students of The Institution who are members of the Glasgow and District Association of The Institution and are British subjects of Scottish parentage.

ROAD ABSTRACTS.

The publication of "Road Abstracts," compiled by the Department of Scientific Research and the Ministry of Transport, is being continued in 1941. By arrangement with the Institution of Municipal Engineers, members are enabled to subscribe for these Abstracts at the rate of 8*s*. 6*d*. per annum (postage free)—one-half the usual rate charged.

All subscriptions run from January.

PARSONS MEMORIAL LECTURE, 1940.

The Parsons Memorial Lecture for 1940, entitled "The Engining of Highly-Powered Ships," will be delivered by Sir Stephen Joseph Pigott, D.Sc., M. Inst. C.E., in the Lecture Theatre of the Literary and Philosophical Society, Newcastle-on-Tyne, on Tuesday, 17 December, 1940. The time for the Lecture to commence has been provisionally fixed at 6 p.m.

NATIONAL PHYSICAL LABORATORY.

The Council have nominated Mr. W. J. E. Binnie, Past-President, as the second representative of The Institution on the General Board of the National Physical Laboratory, in succession to Sir Richard Redmayne, for 6 years from December, 1940.

MINISTRY OF HOME SECURITY, RESEARCH AND EXPERIMENTS DEPARTMENT.

A limited number of copies of the undermentioned Bulletins, which have been issued by the Research and Experiments Department of the Ministry of Home Security, are available to members upon application to the Secretary of The Institution. Application should be made by post-card, quoting the Bulletin No. given in the left-hand column.

Bulletin No.	
C.1.	New Design Methods for Strutting of Basements, Etc.
C.2.	Consolidation of Earth Covering on Anderson Shelters.
C.3.	The Propping of Reinforced Concrete Beams.
C.4.	The Protection of Glass in Hospitals.
C.5.	Steps that should be taken to increase the Resistance of "Umbrella" type Shed Roofs to Collapse due to Air Attack.
C.6.	Damage to Cast Iron Pipes in Works.
C.7.	The Protection of Factory Glazing.
C.8.	Structural Damage Caused by Recent Air Raids to Some Single-Storey Buildings.
C.9.	The Protection of Plate Glass Windows.
C.10.	Flexible Substitutes for Glass.
C.11.	Chemical Fire Extinguishers. Their application to incendiary bombs and resultant fires.
C.12.	Single-Storey Wartime Factory Design.
C.13.	Obscuration, Ventilation, and Protection from Glass in Large Buildings.

COMMITTEES OF THE COUNCIL FOR 1940–41.

The President is *ex officio* a member of all Council Committees.

Executive Committee, War Period.

The President.

The Vice-Presidents.

Asa Binns.
S. B. Donkin.
W. T. Halcrow.
R. G. Hetherington.
Sir Leonard Pearce.
Sir Clement Hindley.

With power to co-opt.

Standing Committees.

Education and Training.

Chairman.—Professor C. E. Inglis.

Professor Gilbert Cook.
C. G. Du Cane.
T. P. Frank.
Ralph Freeman.
Dr. W. H. Glanville.
Sir Leonard Pearce.
H. C. Whitehead.

Board of Moderators.—Professor Inglis, Professor C. L. Fortescue, Professor B. P. Haigh, Professor A. J. Sutton Pippard, Dr. R. E. Stradling, Professor W. N. Thomas.

Finance.

Chairman.—Asa Binns.

Sir John Thornycroft.
J. R. Davidson.
T. P. Frank.
W. T. Halcrow.
R. G. Hetherington.
W. H. Morgan.
V. A. M. Robertson.
W. J. E. Binnie.
S. B. Donkin.
Sir Alexander Gibb.
Sir Clement Hindley.

General Purposes.

Chairman.—The President.

Dr. David Anderson.
Professor C. E. Inglis.
Sir John Thornycroft.
F. E. Wentworth-Sheilds.
Asa Binns.
Raymond Carpmael.
J. R. Davidson.
T. P. Frank.
W. T. Halcrow.
R. G. Hetherington.
W. H. Morgan.
W. J. E. Binnie.
S. B. Donkin.
Sir Alexander Gibb.
Sir Clement Hindley.

Membership.

Chairman.—Dr. David Anderson.

Sir Athol Anderson.
C. G. Du Cane.
R. G. Hetherington.
R. F. Hindmarsh.
R. J. M. Inglis.
W. H. Morgan.
A. G. Vaughan-Lee.

Mr. M. F-G. Wilson has been asked to attend meetings of this Committee.

Professional.

Chairman.—The President.

Dr. David Anderson.
Professor C. E. Inglis.
Sir John Thornycroft
F. E. Wentworth-Sheilds.
Sir Harley Dalrymple-Hay.
C. G. Du Cane.
Ralph Freeman.
Col. C. L. Howard Humphreys.
R. J. M. Inglis.
A. G. Vaughan-Lee.
H. C. Whitehead.
W. J. E. Binnie.
S. B. Donkin.
Sir Alexander Gibb.
Sir Clement Hindley.

Mr. M. F-G. Wilson has been asked to attend meetings of this Committee.

Publications and Library.

Chairman.—F. E. Wentworth-Sheilds.

Sir Athol Anderson.
Asa Binns.
Raymond Carpmael.
Dr. W. H. Glanville.
W. T. Halcrow.
Sir Leonard Pearce.
A. G. Vaughan-Lee.

Institution Committees.

Local Associations.

Chairman.—Sir John Thornycroft.

Sir Athol Anderson.
Professor Gilbert Cook.
Sir Harley Dalrymple-Hay.
R. F. Hindmarsh.
Col. C. L. Howard Humphreys.
V. A. M. Robertson.
H. C. Whitehead.

With the Chairmen of the Local Associations, one other member of each Local Association of Corporate Members and Students in the British Isles, and the Chairmen or any members of the Overseas Advisory Committees who may be in England.

Overseas Sub-Committee :—The Chairman and Members of Council on the Local Associations Committee, the Chairmen of Overseas Local Associations and the Chairmen or Members of Overseas Advisory Committees who may be in England.

Public Relations Committee.

Chairman.—W. T. Halcrow.

F. E. Wentworth-Sheilds.
Hugh Beaver.
Dr. Herbert Chatley.
J. R. Davidson.
Conrad Gribble.
Col. C. L. Howard Humphreys.
R. W. Mountain.
C. M. Norrie.
S. R. Raffety.
V. A. M. Robertson.
J. E. Swindlehurst.
W. A. Tookey.
M. T. Tudsbery.
D. M. Watson.

Railway Engineering Section Committee.

Chairman.—Raymond Carpmael.

F. E. Wentworth-Sheilds.
George Ellson.
W. T. Halcrow.
R. J. M. Inglis.
V. A. M. Robertson.
W. K. Wallace.
With others to be appointed.

Research.

Chairman.—Dr. David Anderson.

Professor C. E. Inglis.
F. E. Wentworth-Sheilds.
Alfred Bailey.
G. M. Burt.
Raymond Carpmael.
F. C. Cook.
Professor Gilbert Cook.
Ralph Freeman.
Dr. W. H. Glanville.
W. T. Halcrow.
R. G. Hetherington.
R. J. M. Inglis.
Professor A. H. Jameson.
Professor F. C. Lea.
Professor K. N. Moss.
Sir Leonard Pearce.
Professor A. J. S. Pippard.
Dr. S. L. Smith.
R. H. H. Stanger.
Dr. R. E. Stradling.
W. J. E. Binnie.
S. B. Donkin.
Sir Clement Hindley.
M. F-G. Wilson.

Deterioration of Structures Exposed to Sea Action.

Chairman.—W. T. Halcrow.

F. E. Wentworth-Sheilds.
Sir Athol Anderson.
W. P. Barron.
Raymond Carpmael.
C. G. Du Cane.
Dr. S. A. Main.
Sir Cyril Kirkpatrick.
M. F-G. Wilson.

Road Engineering Section Committee.

Chairman.—F. C. Cook.

F. E. Wentworth-Sheilds.
H. E. Aldington.
Professor R. G. H. Clements.
Arthur Floyd.
T. P. Frank.
Dr. W. H. Glanville.
W. H. Morgan.
Dr. R. E. Stradling.

TRANSFERS, ELECTIONS, AND ADMISSIONS.

Since the 24th September, 1940, the following elections have taken place:

Meeting.	*Member.*	*Associate Members.*
19 November	1	63

and during the same period the Council have transferred three Associate Members to the class of Members, and have admitted forty-four Students.

DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations:—

Deaths.

MORGAN, Sir Charles Langbridge, C.B.E., D.Eng. (E. 1904. T. 1919.)	*Past-President.*
BALFOUR, John Aylmer. (E. 1904. T. 1919)	*Member.*
CLARK, Robert George, O.B.E. (E. 1905. T. 1926.)	,,
GRIFFITH, Percy. (E. 1892. T. 1901.)	,,
HENRIQUES, Philip Quixano. (E. 1917. T. 1933.)	,,
NEWINGTON, Frank Alfred. (E. 1894. T. 1902.)	,,
PAWLEY, Richard. (E. 1883. T. 1889.)	,,
BOYES, Frederick Thomas. (E. 1919.)	*Associate Member.*
*COLBOURNE, Jack Herbert, B.Sc. (E. 1932.)	,, ,,
CONNER, Benjamin. (E. 1897.)	,, ,,
RIGG, Robert James. (E. 1940.)	,, ,,
*EVANS, Douglas Alban St. John, B.Sc. (A. 1938.)	*Student.*
*LUPTON, John Sheppard Hewitt. (A. 1939.)	,,
*MARTIN, Francis Robert. (A. 1934.)	,,

* Killed on active service.

Resignations.

BLACKWELL, John Eaton, D.S.O., B.A. (E. 1905. T. 1920.)	*Member.*
DORMER, Henry Duncan. (E. 1902. T. 1920.)	,,
BARBER, Cecil. (E. 1909.)	*Associate Member.*
LAY, Robert Arthur. (E. 1913.)	,, ,,
MACLACHLAN, George. (E. 1904.)	,, ,,
MASON, Arnold Stewart. (E. 1905.)	,, ,,
SHARPE, Henry Grattan, B.A., B.A.I. (E. 1909.)	,, ,,
LUFFLEY, Alfred Edward. (E. 1925.)	,, ,,
MASSIE, James Crichton. (A. 1936.)	*Student.*

RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

AIRCRAFT. NEEDHAM, C. H. L. "Aircraft Design. Vol. 2. Aerostructures." 1939. Chapman and Hall. 18*s*.

See also FLIGHT; GLIDING; INSPECTION.

AIR DEFENCE. MINISTRY OF HOME SECURITY. A.R.P. Memo. No. 16. "Emergency Protection in Factories." 1940. H.M.S.O. 1*d*.

—— * ASSOCIATION OF ARCHITECTS, SURVEYORS, AND TECHNICAL ASSISTANTS. Drawings for "Haldane" Bombproof Shelter. 1940. 113, High Holborn, W.C.2. 5*s*. 8*d*.

BUILDING. *WILLIAMS-ELLIS, C. "Cottage Building in Cob, Pisé, Chalk, and Clay." 1919. Newnes. 6*s*.

CEMENT. CRADDOCK, Q. L. "Cement Chemists' and Works Managers' Handbook." 1940. Concrete Publications. 15*s*.

CLAYWORKING. BROWN, A. E. "Practical Clayworking." 1936. Clayworker Press. 7*s*. 6*d*.

COAL-DUST ENGINES. WENTZEL, W. "The Ignition and Combustion Process in the Coal-Dust Engine." 1932. Fuel Publications. 2*s*. 6*d*.

COMPRESSION-IGNITION ENGINES. DICKSEE, C. B. "The High-Speed Compression-Ignition Engine." 1940. Blackie. 16*s*.

CUTTING TOOLS. KURRIM, M., and LEA, F. C. "Cutting Tools for Metal Machinery." 1940. Griffin. 16*s*.

ELECTRICITY. LAWS, E. "Electricity applied to Marine Engineering." 1940. Inst. Marine Engineers. 5*s*. 6*d*.

ELECTRICITY-MOTORS. HALL, J. G. "Small Electric Motor Construction." 1940. Marshall. 2*s*.

EXPLOSIONS. MINES DEPARTMENT. "Explosion at Valleyfield Colliery, Culross, Fife. Report." 1940. H.M.S.O. 1*s*. 6*d*.

FLIGHT. KERMODE, A. C. "Flight without Formulæ." 1940. Pitman. 6*s*.

FOUNDATIONS. PLUMMER, F. L., and DORE, S. M. Soil Mechanics and Foundations." 1940. Pitman Publishing Corp'n. 27*s*.

GEOLOGY. LEE, J. S. "The Geology of China." 1939. Murby. 31*s*. 6*d*.

GLIDING. BARRINGER, L. B. "Flight without Power." 1940. Pitman Publishing Corp'n. 23*s*.

INSPECTION. PARKINSON, A. C. "Engineering Inspection . . . including special reference to Aero requirements." 1940. Pitman. 6*s*.

LIFTS. GOOCH, L. J. "Electric Lifts." 1940. Marryat and Scott, Ltd. 10*s*. 6*d*.

LOCOMOTIVE VALVES. LAKE, C. S., and REIDINGER, A. "Locomotive Valves and Valve Gears." 1940. Marshall. 5*s*.

MARINE ENGINEERING. *See* ELECTRICITY.

MOSQUITO CONTROL. MINISTRY OF HEALTH. "Memorandum on Measures for the Control of Mosquito Nuisances in Great Britain." 1940. H.M.S.O. 6*d*.

PRODUCTION. JONES, E. J. H. "Production Engineering: Jig and Tool Design." 1940. Newnes. 12*s*. 6*d*.

RAILWAYS. BARRIE, D. S., and LEE, C. E. "The Sirhowy Valley and its Railways." 1940. Railway Publishing Co. 3*s*. 6*d*.

ROADS. O'DWYER, S. "The Roman Roads of Wales." 1934–1937. Montgomery Printing Co., Newtown. Parts 1–6. 9*s*. 6*d*. [Part 2 is out of print.]

ROPES AND ROPEWAYS. CARTER, H. R. "Rope, Twine, and Thread Making." 1924. Bale. 7*s*. 6*d*.

—— ILSLEY, L. C., and MOSIER, M. "Inspection and Maintenance of Mine Hoisting Ropes." 1940. Supt. of Documents, Washington. 5 cents.

SOIL MECHANICS. PLUMMER, F. L., and DORE, S. M. "Soil Mechanics and Foundations." 1940. Pitman Publishing Corp'n. 27*s*.

STEAM TABLES. BRITISH ELECTRICAL AND ALLIED MANUFACTURERS' ASSOCIATION. "The 1940 Heat Drop Tables." 1940. Arnold. 15*s*.

TECHNICAL TERMS. HUSAIN, S. Z. "A List of Technical Terms. (In English, Urdu, Marathi, and Hindi)." 1939. Milford. 4*s*.

TELEPHONY. HERBERT, T. E., and PROCTER, W. S. "Telephony. Vol. 1, Manual Switching Systems and Line Plant. Vol. 2, "Automatic Telephony." 1934 and 1938. Pitman. 20*s*. each.

(* *The foregoing books, with the exception of those marked with an asterisk, may be borrowed from the Loan Library.*)

ABSTRACT OF A PAPER FOR DISCUSSION.

The following Paper will be brought forward for discussion on the date indicated in the margin of the abstract, and will be published, with reports of the oral and written discussions upon it, in the Journal. Members desiring to take part in the consideration of this Paper should apply forthwith for advance copies, which will be forwarded as soon as they are ready. Applications for proofs should be made on postcards, quoting the number of the Paper.

A period of about 3 months from the date of publication of the Paper in the Journal is generally allowed for written communications, which should be :—

(*a*) As concise as possible and entirely relevant to the subject-matter of the Paper ;

(*b*) Written legibly or typed with the lines openly spaced.

Paper No. 5255.

Date of Discussion 21 Jan.

"The Design of Sewage-Purification Works."

By Herbert Cecil Whitehead, M. Inst. C.E.

The Author emphasizes the complications of design due to the conflicting character of some of the many natural forces which may be usefully employed, and urges the necessity for an orderly sequence of operations in separating the various kinds of impurities.

He outlines the requirements for inland towns, recommends the extension of partial treatment for storm-water, and considers the factors affecting the oxidizing capacity of percolating filters and describes recent variations in methods of operation.

Suggestions are made for improvement of the activated-sludge process, the design of humus tanks is considered, and a brief review is given of sludge treatment and disposal.

LOCAL ASSOCIATIONS.

CHAIRMEN AND HONORARY SECRETARIES.

The names of Chairmen and the names and addresses of the Honorary Secretaries of Local Associations are :—

In the British Isles.

Birmingham. *Chairman.*—B. C. Hammond, M. Inst. C.E. *Hon. Secs.*—S. J. Davies, Assoc. M. Inst. C.E., P.W. Dept., Dell Road, Cotteridge, Birmingham 30, and H. L. Price, Stud. Inst. C.E., 63 Middleton Hall Road, King's Norton, Birmingham 30.

Bristol. *Chairman.*—A. M. Paterson, M.C., M. Inst. C.E. *Hon. Sec.*—T. B. Cooper, B.Sc., Assoc. M. Inst. C.E., Refuge Assurance Buildings, Bristol 1.

Edinburgh. *Chairman.*—J. G. MacGregor, M. Inst. C.E. *Hon. Secs.*—J. S. McNeil, B.Sc., Assoc. M. Inst. C.E., Ministry of Transport, Castle Terrace, Edinburgh 1, and D. A. Coode, Stud. Inst. C.E., Orwell Lodge, Juniper Green, Midlothian.

Glasgow. *Chairman.*—Professor Gilbert Cook, D.Sc., F.R.S., M. Inst. C.E. *Hon. Secs.*—William MacGregor, B.Sc., Assoc. M. Inst. C.E., Engineering Dept., The University, Glasgow, W.2, and W. M. Cormie, B.Sc., Stud. Inst. C.E., Glenfield, Dumbuck Crescent, Dumbarton.

Newcastle-upon-Tyne. *Chairman.*—P. A. R. Leith, M. Inst. C. E. *Hon. Secs.*—W. M. Anderson, Assoc. M. Inst. C.E., Civil Engr.-in-Chief's Dept., H.M. Dockyard, Chatham, and T. Storer, Stud. Inst. C.E., Victoria Buildings, Stockton-on-Tees. (Tees-side Branch.)

North-Western. *Chairman.*—R. D. Brown, M. Inst. C.E. *Hon. Secs.*—R. J. Cornish, M.Sc., Assoc. M. Inst. C.E., Municipal Engineering Dept., College of Technology, Manchester 1, and J. Dann, M.Sc. Tech., Stud. Inst. C.E., Mill Farm, Green Lane, Baguley, Manchester.

Northern Ireland. *Chairman.*—R. E. L. Clarke, B.A., B.E., Assoc. M. Inst. C.E. *Hon. Secs.*—S. H. W. Middleton, B.A., B.A.I., Assoc. M. Inst. C.E., Water Office, Royal Avenue, Belfast, and G. F. Chambers, B.Sc., Stud. Inst. C.E., 60 Somerton Road, Antrim Road, Belfast.

South Wales and Monmouthshire. *Chairman.*—James Hassall, M. Inst. C.E. *Hon. Secs.*—H. R. Reynolds, Assoc. M. Inst. C.E., 27 Roath Court Place, Cardiff, and J. W. Cotterill, Stud. Inst. C.E., 124 Redlands Road, Penarth, Glam.

Southern. *Chairman.*—J. Parkin, M. Inst. C.E. *Hon. Secs.*—P. E. Sleight, M. Eng., Assoc. M. Inst. C.E., The Municipal College, Portsmouth, and M. C. Privett, Stud. Inst. C.E., 59 Southampton Road, Fareham, Hants.

Yorkshire. *Chairman.*—A. G. Beaumont, M. Inst. C.E. *Hon. Sec.*—J. H. W. Freeman, B.Sc., Assoc. M. Inst. C.E., Briarcliffe, Harrogate Road, Alwoodley, Leeds.

OVERSEAS.

BUENOS AIRES. *Chairman.*—H. A. MCGILLYCUDDY, B.E., M. Inst. C.E. *Hon. Sec.*—H. W. Stevens, B.Sc., Assoc. M. Inst. C.E., Guanahani 322, Buenos Aires.

MALAYAN. *Chairman.*—A. R. FYFE, M. Inst. C.E. *Hon. Sec.*—J. T. Chester, Assoc. M. Inst. C.E., P.W.D., Singapore, S.S.

SHANGHAI. *Chairman and Acting Hon. Sec.*—N. N. MAAS, B.Sc., M. Inst. C.E., Whangpoo Conservancy Board, Shanghai, China.

VICTORIAN. *Chairman.*—A. C. LEITH, M.C.E., M. Inst. C.E. *Hon. Sec.*—R. G. Knight, M.C., M.C.E., M. Inst. C.E., 53–55 Collins Place, Melbourne, Victoria.

WEST INDIES. *Chairman.*—S. R. H. BEARD, M. Inst. C.E. *Hon. Sec.*—C. L. Champion, B.Sc., Assoc. M. Inst. C.E., c/o Senior Naval Officer, Port-of-Spain, Trinidad, B.W.I.

Synopses of CHAIRMEN'S ADDRESSES.

BIRMINGHAM AND DISTRICT ASSOCIATION.

Meeting, 31 *October,* 1940.

Mr. B. C. Hammond expressed thanks for his election, and recalled his connexion with The Institution as a Student for 34 years. He emphasized the difficulties inevitable in war-time, and the necessity for all members to continue their interest in The Institution and the Association. The magnitude of the national issues at stake was reflected in all discussions of engineering topics. He remarked upon the antiquity of civil engineering in general, and road-making in particular, recalling the achievements of the Egyptians, Phœnicians, Greeks, and Romans, and deploring the insignificant historical knowledge that remained of the actual methods employed. He reviewed the records of Roman achievements in road-making in Britain, and observed that the work of earlier peoples was still discernible, in comparison with Roman military design. The principal Roman routes, had they been more fully preserved until to-day, would have served as the major lines of a system of motorways suitable for the most modern needs.

Little engineering progress was made from the Roman evacuation of Britain until road communications began to be improved in the 18th century. The association of the name of the first President of the Institution, Thomas Telford, with road-making led to thoughts of earlier pioneers, such as General Wade, John Metcalfe, and John Loudoun Macadam. The era of these experts was followed by a period of railway development, when not only were roads relegated to obscurity, but also the cumulative effect of some decades of legislation favoured various public utility undertakings, and thus added complication and expense under modern con-

ditions of traffic. Mr. Hammond pleaded for the systematic overhaul and simplification of legislation relative to highway matters as an essential to post-war reorganization and re-planning. He hoped for a concentration of the efforts which would be necessary for recovery, and envisaged the possible future administrative machinery, and the further nationalization of highways.

He reviewed the advantages of special routes exclusively for fast mechanical traffic, and mentioned certain defects still inherent in even the latest pre-war reconstruction practice, stressing the necessity for a careful decision by the State definitely for or against motor-roads as an essential preliminary to re-planning. He observed that future practice would be uncertain until these and other post-war conditions were known, and indicated the manner in which future amenities and public safety might require to be studied. He emphasized the importance of skilful repair work during the next few years, and discussed methods of construction and workmanship and the use of plant.

From the earliest times river crossings had had an important effect upon the establishment of towns and cities. Mr. Hammond considered that actual principles of construction hitherto accepted would probably be less affected than new road construction by actual conditions, since the main problem would probably be the selection of suitable types at an economic cost. He discussed points of design, and pleaded for simplicity, citing examples of its effect. He reviewed recent developments in the principles of bridge construction, including the use of portal-frames, electric welding, and vibrated concrete.

Referring to the reconstruction and restoration of old structures, he mentioned that many weak, narrow, or otherwise dangerous bridges existed prior to the outbreak of war, and their number would inevitably be augmented by war conditions. The interesting practices to be found in repair and reconstruction work were exemplified by work carried out by Mr. Hammond during recent years on four bridges in which all the conditions were entirely dissimilar. Two of these had been originally designed and built by Telford more than 100 years ago.

Whatever future conditions might prove to be, the activities of members of The Institution would remain as defined by its Charter, namely, " the art of directing the great sources of power in Nature for the use and convenience of man."

YORKSHIRE ASSOCIATION.

Meeting, 12 *October*, 1940.

Mr. A. G. Beaumont recalled that the subject of the previous Chairman's address had been " The Engineer—Technically, Commercially and Personally ", and that the President of The Institution had dealt with the development of the status of the Civil Engineer. In the present circum-

stances it seemed natural for him to discuss "Civil Engineers and the War", with some reference to waterworks engineering and administration.

Many of our members were serving in the Forces, whilst others were engaged in special war work, or in maintaining public services; and the Central Register permitted engineers with special qualifications to be selected for important national work. Further the importance of the civil engineer was recognized by placing him in the Schedule of Reserved Occupations at the age of 23; moreover a Student under that age had special facilities for completing his studies; and The Institution's by-laws could be relaxed in order to avoid hardship.

Modern warfare had become mechanized to such an extent that engineers had been blamed for making it possible. Probably no one had contributed more than the engineer to the development of civilization, and it was a calamity that his work should be used for wholesale destruction. Mr. W. J. E. Binnie's address in 1938 had mentioned that 450 years ago Leonardo da Vinci had suppressed his design for a submarine because of its sinister possibilities.

To meet the German "Blitzkrieg" a still more formidable war machine had to be produced, and every civilian engineer must fight with the tools of his craft. The normal construction of public works and improvements was suspended to a great extent, and our workshops, shipyards, and man-power had to be turned over to war work, and new munition works and aerodromes constructed. A vast civil defence organization had become necessary, and the provision of shelters had created new problems.

Mr. Beaumont emphasized that in all future engineering designs the recurrence of aerial warfare would have to be taken into account. He thought that, where possible, greater attention would be given to the choice of site, and that there would be a general tendency towards decentralization: this might apply to industrial works, electrical generating-stations, waterworks pumping-plants, gasworks, and other public utility and transport undertakings. Interruption of services might be reduced by having smaller units, more stand-by plant and facilities for repairs, and interconnexions and alternative routes for mains, cables, and tracks.

Experience had shown that a framed structure with light panels which could yield before transmitting blast pressure was more likely to survive than the older type of walled building. The fracture of one member of a frame should not cause its failure, or spread the collapse to adjoining work. Further investigation was required as to the comparative resistance to blast and fire of steel framework, reinforced concrete, and other forms of construction.

Turning to waterworks topics, Mr. Beaumont observed that, after earlier experience on constructional work, the engineer usually became involved in administrative duties. Whilst the constant supply of pure water was the first consideration, an undertaking must pay its way and

the cost must be reasonable—about 3*d*. or 4*d*. per ton delivered to the premises—for the public was inclined to regard water as one of nature's bounties.

He referred to the volume of recent legislation, special and general, affecting waterworks, and relating to public health, land drainage, factories, fire brigades, access to mountains, county areas, and the safety of reservoirs, and also to the Water Undertakings Bill, which had been withdrawn when war occurred. The Ministry of Health had displayed a closer interest in water-supply, and the Croydon epidemic had led them to urge various precautions, which included the medical examination of workmen to detect potential typhoid-carriers. The treatment and sterilization of water was already widely practised, but since the outbreak of war sterilization had been insisted upon.

"A.R.P." had become merged in "Civil Defence." A year or two before the war engineers had been requested to take precautions for ensuring the continuity of supplies in case of attack, and much practical work had been done, whilst the Government had made grants towards certain approved measures. Major works such as reservoirs, pumping-stations, and filters presented special problems; more obvious measures included decentralizing stores, increasing stocks of material and equipment, improving mains, linking-up with neighbouring authorities, co-operation in repair gangs, emergency organization and training of personnel, and the guarding of vulnerable points.

The consumption of water had declined in evacuated areas, but had increased in many industrial towns, whilst populations transferred to rural districts also required additional water and other services. The severe winter of 1939–40 had caused great inconvenience and waste from burst pipes, whereas the hot summer had led to heavy demands, and many authorities had imposed restrictions in order to conserve supplies in case of war emergencies. The influence of these factors, including the variations in consumption during night "alerts", was illustrated by diagrams.

In conclusion, Mr. Beaumont said that we looked forward with confidence to the successful conclusion of the war. It was impossible to foresee what the political and economic conditions might be, but in reconstruction—in the widest sense of the term—the engineer would again have the duty and privilege of working "for the use and convenience of man."